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MINUTES OF PROCEEDINGS

OF

THE INSTITUTION

OF

CIVIL ENGINEERS.

VOL. CCXVI.

EDITED BY

H. H. JEFFCOTT, Sc.D., B.A.I., Assoc. M. Inst. C.E., Secretary.

LONDON:
Published by The Institution.
GREAT GEORGE STREET, WESTMINSTER, S.W. 1.
[Telegrams, "Institution, London." Telephone, "Victoria, 4677."]
1923.

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THE SECRETARY,
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Great George Street, Westminster, S.W. 1.

LONDON: PRINTED BY WILLIAM CLOWES AND SONS, LIMITED,
DUKE STREET, STAMFORD STREET, S.E. 1, AND GREAT WINDMILL STREET, W. 1.

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CORRIGENDA.

Vol. ccii, p. 235, line 6 from bottom, *for* " ,, " *read* "sq. miles."
,, ,, ,, ,, 5 ,, ,, *for* " ,, " *read* "feet."

THE INSTITUTION OF CIVIL ENGINEERS.

SESSION 1922–1923.—PART II.

MINUTES OF PROCEEDINGS.

6 February, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The **President** in announcing the death of Mr. Alexander Ross, stated that the Council had passed the following resolution, in which, he was sure, the meeting would concur :—

" That the Council record the deep regret with which they have learned of the death of Mr. Alexander Ross, a Past-President of The Institution, whose association with it as a Member for nearly 43 years was marked by the warmest interest in its well-being, and desire to convey their sincere condolences to the members of his family."

The Council reported that they had recently transferred to the class of

Members.

Ernest James Buckton, B.Sc. (Eng.) (*Lond.*).
Henry Ralph Crabb.
William Thomas David, M.A. (*Cantab.*), Sc.D. (*Wales*).
Robert Wilson Dron.
David Edwards.
Hugh Gibson.
Shirley Onslow Limby.
Professor **Ernest Romney Matthews.**
David Chalmers Urie.
Hugh Pass Williamson, *M.C.*

And had admitted as

Students.

Robert Clive Beecroft, B.Eng. (*Sheffield*).
Alfred Henry Cantrell.
Richard Godfrey Arthur Colley.
St. John Coventry.
Thomas Burnaford Davey, B.Sc. (Eng.) (*Lond.*).
Colvin Edward Docker.
George Frank Garnett.
Santosh Kumar Ghosh, B.Sc. (*Edin.*).
James Kenneth Grant, B.Sc. (*Edin.*).
Charles Henry McLeod Hawk.
James McEwen King.
Vishwanath Duyanashwar Kothare, B.Sc. (*Bombay*).
George Taylor Leithhead, B.Sc. (*Edin.*).

Students—continued.

Jacobus Lessing.
Owen Cadwalader Llewelyn-Jones.
John Long.
Edwin George McLeod.
Frederick Rankin MacRae.
Edwin Hope Marfleet.
Kenneth George Hope Montgomery-Smith, B.Sc. (Eng.) (*Lond.*).
James Roland Phillips, B.Sc. (Eng.) (*Lond.*).
William Edward Puddy.
Edward Joseph Pyke, B.A. (*Cantab.*).
Robert Russell Harding Rains.
Harcourt Stuart Roe.
Ninham Shand, B.Sc. (*Cape*).
Lachhman Singh.
Cecil Raymond Stone, B.A. (*Cantab.*).
Frederick William Sully.
Herbert Ronald Tasker.
George Ivan Bickerdyke Thomas.
Alan Stanley Watson.
Alfred Obré Whitehouse.

The Scrutineers reported that the following candidates had been duly elected as

Members.

James Andrew.
John Bowden.
James Cracroft Haller.
Malcolm Hunter Logan, *M.C.*
Alexander Macdonald, O.B.E., *M.C.*
Edward Giles Stone.
George Ward Wright.

Associate Members.

George Patterson Alexander.
James Anderson, Junr., Stud. Inst. C.E.
Donald Henry Currer Briggs, M.A. (*Oxon.*).
Cyril Stapley Chettoe, B.Sc. (Eng.) (*Lond.*).
Edward Crowther, M.Eng. (*Liverpool.*)
Errol Raffael Henry Darwin, B.Sc. (*Adelaide*).
Michael Bruce Urquhart Dewar, M.A. (*Cantab.*).
Edward Holmes Elgar.
Nelson Norbert Farrell.
Alfred Reginald Goldthorp, Stud. Inst. C.E.
Geoffrey Handley Hopewell.
Edward Harold Hutton, Stud. Inst. C.E.
Leonard Frank Jeffrey, B.Sc. (Eng.) (*Lond.*).
Ragnvald Jensen.
Justin McCarthy, *M.C.*, Stud. Inst. C.E.
Gilbert McIntosh, B.Sc. (*Glas.*), Stud. Inst. C.E.
Charles Calder Mackintosh, B.Sc. (*Glasgow*).
Frederick William Munro.
Herbert John Nichols, B.Sc. (Eng.) (*Lond.*).
Robert Blair McCabe Potter, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Walter Herbert Price, Stud. Inst. C.E.
Alec John Ryde.
Gerard William Sampson.
Alfred George Scott, B.Sc. (*Dundee*).
William Robert Sheffield.
Donald Arthur Stevens, B.Sc. (Eng.) (*Lond.*).
Alfred Leslie Taylor, B.Sc. (Eng.) (*Lond.*).
Frederick George Turner, B.Sc. (Eng.) (*Lond.*).

(*Paper No. 4363.*)

"Wind-Pressures, and Stresses caused by the Wind on Bridges."

By Douglas Henry Remfry, Assoc. M. Inst. C.E.

The Indian bridge rules specify that the allowance for wind-pressure should be 56 lbs. per square foot on unloaded structures and 33⅓ lbs. per square foot on loaded structures. There is a general tendency now to consider these pressures as being too high. The conclusions arrived at by the Author are that a very considerable reduction in the wind-pressure on unloaded spans might be allowed, but that very slight reductions, if any, should be permitted on loaded structures, below the values specified. A further conclusion is that the allowance for wind-pressure on bridges for narrow-gauge railways should not differ substantially from that for 5-foot 6-inch gauge.

It has been usual in the British Isles to take the maximum wind-pressure on an unloaded structure as 56 lbs. per square foot of surface exposed. This high value was thought necessary when the profession was greatly disturbed by the Tay Bridge disaster (28 December, 1879). The result has been very high expenditure on large bridges. For instance, in designing the Forth Bridge the following stresses were estimated on a bottom boom of one of the cantilevers:—

	Tons.
Stress due to dead load	2,282
,, ,, live load	1,022
,, ,, wind load	2,922
Total	6,226

These wind stresses are nearly three times as great as the live-load stresses, having been calculated on a wind-pressure of 56 lbs. per square foot. Such a wind-pressure seems too high and leads to unnecessary cost. In the new Quebec Bridge the allowance for wind-pressure was reduced to 30 lbs. per square foot.

There is no doubt that the Committee appointed soon after the Tay disaster to consider the question of wind-pressure on railway structures was incorrectly informed as to the velocities corresponding to the records obtained in England with the Robinson anemometer.

It was usual to assume that the actual velocity of the wind was given by multiplying the velocity of the revolving cups by 3; but in 1902 it was proved that the correct coefficient is 2·2. The highest actual mean velocities of about 75 miles per hour had, therefore, been wrongly taken as velocities of over 100 miles per hour, and maximum gust velocities of about 100 miles per hour had been exaggerated to 135 miles per hour. Further, at the time of the Tay Bridge disaster, the coefficient used in the formula connecting pressure with velocity had not been established with such accuracy as at present. and it was generally assumed to be somewhere about 0·0035. Dr. T. E. Stanton has recently shown, however, that the correct formula is $P = 0{\cdot}0031\ V^2$, where P denotes pressure in pounds per square foot and V denotes velocity in miles per hour.[1]

The experiments carried out at the Forth Bridge, although very instructive, led to an exaggerated idea of wind-pressures. It has since been shown that the pressure-boards used had a large factor of error owing to inertia, and that the pressures recorded were much too high. Probably the best instrument is an anemometer of the Dines type, which records not only the average wind-velocity, but also the velocity in gusts. It is possible that it does not indicate the absolute maximum velocity of the gusts, owing to its dead-beat action; but it gives much valuable information which cannot be obtained from instruments of other types.

It is proposed to consider the question under the following subheads :—

(1) Maximum mean wind-velocities and the rise of the velocity in gusts.
(2) The fact that the conditions in India are more severe than in England.
(3) The dynamic action of the gusts.
(4) The pressure as affected by the size of a structure.
(5) The effect of the shape of a structure on the pressure.
(6) The shielding action due to double bars and double girders.
(7) The effect of rain with wind.
(8) The reduction in pressure which may be allowed when considering a loaded span.
(9) Methods of calculating wind stresses in bridges and the approximate range of stress caused in such structures by the wind.

---

[1] Minutes of Proceedings Inst. C.E., vol. clvi, p. 78.

MAXIMUM VELOCITIES.

Maximum Mean Velocities.—Taking fourteen important stations in the British Isles for a period of 14 years from 1890 to 1904, the maximum mean velocity recorded only reached the following values :—

77 to 80	miles	per hour on	one occasion,
73 to 77	,,	,,	one occasion,
70 to 73	,,	,,	five occasions,
65 to 69	,,	,,	five occasions.

The records for seven other stations in the British Isles for the 14 years between 1899 and 1913 showed only on two occasions a maximum mean velocity reaching 64 to 75 miles per hour. At the time of the Loch Leven accident, when a train was blown over by the wind, the velocity reached was 71 miles per hour, and 73 miles per hour was indicated at Barrow, some little distance away. At the time of the Tay disaster the mean velocity recorded at Glasgow was 52 miles per hour, while at Aberdeen at the same time 70 miles per hour was registered for 5 minutes. It may be assumed that 75 miles per hour is the maximum mean wind-velocity in the British Isles, which is practically never exceeded.

Maximum Velocity in Gusts.—The variation of the wind-velocity in gusts above and below the mean velocity depends to a certain extent upon the exposure of the situation. It would appear that the more exposed the situation is, the less the variation from the mean will be. It is probable, moreover, that as the height of the point of observation above the surface of the earth increases the variation in gusts lessens, although the mean velocity in itself would naturally increase. The Author has been given to understand that, in flying, the most dangerous height is between 150 and 250 feet above the ground. Above 250 feet, flying appears to be much safer, owing, probably, to less variation in the velocity due to gusts. Dr. C. G. Simpson, Director-General of Observatories, Simla, has supplied the following approximate formulas connecting the maximum and minimum velocity in gusts with the maximum mean velocity :—

$$\text{Maximum velocity in gusts} = 1{\cdot}3\ V + 1{\cdot}5,$$
$$\text{Minimum velocity in gusts} = 0{\cdot}65\ V - 1{\cdot}0,$$

where V denotes the maximum mean velocity in miles per hour. It is understood that these formulas were developed from a consideration of wind-velocities in the British Isles. The records of twenty-one important stations in the British Isles from 1906 to

1913 show that the maximum gusts recorded were 99 miles per hour once, 90 miles per hour twice, and 85 to 89 miles per hour nine times. At the time of the Tay disaster a gust-velocity of 89 miles per hour was recorded at Glasgow. These are corrected velocities. In the case of the Loch Leven accident, the wind-velocity in gusts must have risen above 100 miles per hour, as the lightest carriages needed a pressure of 32 lbs. per square foot to overturn them. Other carriages required a pressure of 42 lbs. per square foot, which would be equivalent to a maximum gust-velocity of nearly 120 miles per hour; but doubtless these were dragged over by the rest of the train. It may be assumed that the maximum gust-velocity rarely exceeds 100 miles per hour in the British Isles.

WIND-VELOCITIES IN INDIA.

An examination of the records of thirty-four stations in India, which have been kindly supplied by Dr. Simpson, shows that a mean hourly velocity exceeding 60 to 70 miles per hour has not been recorded. The maximum gust-velocity has exceeded 80 miles per hour on seven or eight occasions and has probably reached 100 miles per hour once. In Ceylon a gust-velocity of 88 miles per hour for 3 minutes was recorded at Jaffna on the 18th July, 1916. From these results it would appear that Indian wind-velocities do not differ very widely from those occurring in Great Britain.

In India, however, the difficulty arises that cyclones or tornadoes are likely to occur. Cyclones are very local in their maximum effect, perhaps only affecting a section of the country ½ to ¾ mile wide, and 2 or 3 miles long. It is, therefore, practically hopeless to expect to get records of maximum wind-velocities in cyclones from the observatories, and the only data available are such inferences as can be drawn from trains blown over by the wind and from damage caused to structures. It is probable that much higher velocities occur in cyclones than in ordinary storms. Further, the velocity rises very rapidly, and it has been stated by some observers that considerable dynamic action may take place, accompanied by a large uplifting tendency due to sudden local variations of barometric pressure. There is evidence to show that the maximum wind-intensity in these cyclones sometimes only lasts for a few seconds. The evidence afforded by trains blown over by the wind is very valuable. In a technical Paper on this subject published by the Indian Railway Board seventeen cases are cited. In eleven of these cases sufficient details are given for conclusions to be arrived at with regard to pressures and velocities.

Table I gives particulars of these cases, two of which are abnormal: (1) The accident on the East Indian Railway near Khana Junction, where a mean pressure of 59 lbs. per square foot was needed to upset the average of the whole rake. It is possible that the lightest vehicle upset first, dragging the others after it; but even in that case the vehicles next to the lightest would have helped to hold it up. (2) The other extreme case occurred on the Eastern Bengal State Railway, near the Ghazni Bridge. This was in all respects an extraordinary accident. One loaded wagon, at the end

TABLE I.

Railway.	Locality.	Date.	Pressure Needed to Overturn Vehicle.	Equivalent Pressure on a Flat Plate.	Equivalent Velocity: using Coefficient 0·0031.	
					Steady Wind.	Maximum Gusts.
			Lbs. per Square Foot.	Lbs. per Square Foot.	Miles per Hour.	Miles per Hour.
5-ft. 6-in. gauge E. B. S.	Ichapur	1864	28·0	31·0	100·0	115·5
,,	Arrangghatta	,,	28·3	31·5	100·6	116·0
E. I. R.	Rampur Hat	1902	29·3	32·7	102·0	118·5
,,	Khana Junc.	1874	59·0	64·0	143·0	165·0
Metre gauge E. B. S.	Atrai	1878	26·0	29·0	97·0	111·5
,,	Santahar	1880	31·2	34·6	105·5	121·5
,,	Ghazni Bt.	1881	27·6 63·5	30·6 77·5	99·2 151·5	114·0 174·0
,,	Atrai	1881	20·0 26·0	22·5 29·0	85·0 97·0	98·5 111·5
B. K. R.	Near Bakhshika-Talab	1887	27·4	31·2	100·2	116·0
E. B. S.	Near Madhnagar	1889	17·2	19·2	78·5	91·0
2-ft. 6-in. gauge Khusalgarh-Kohat Tel Railway	Khusalgarh	1902	27·0	30·0	98·0	113·5

In working out the pressure needed to overturn a wagon and the equivalent pressure on a flat plate, in the above list it has been assumed that a wagon would offer a resistance equivalent to 90 per cent. of that offered by a flat plate having the same exposed side area. The maximum gust-velocity has been calculated on an assumption that the maximum gust-pressure on an object the size of a wagon is only 75 per cent. of the gust-pressure on a small plate one foot square due to the wind-stream filaments of maximum velocity. The equation used for connecting pressures and velocities has been taken as $P = 0{\cdot}0031\ V^2$.

of the length overturned, needed a wind-pressure of 63·5 lbs. per square foot to overturn it. It was, no doubt, overturned by the drag of the vehicle behind it. There is evidence to show that one vehicle at least (which would have required 27·6 lbs. per square foot to overturn it) was lifted bodily off the rails, and was deposited, without overturning, in the nullah-bed. An examination of the list, neglecting the two extreme cases, emphasizes the fact that a gust-velocity of at least 110 to 120 miles per hour should be allowed for.

The Southern States of the United States of America are liable to be visited by similar tornadoes. In the case of the St. Louis tornado on the 27th May, 1896, a wind-velocity of 80 miles per hour was recorded for 5 minutes, with an extreme velocity of 120 miles per hour. Apparently, however, the recording-instruments were not in the actual track of the storm. From the damage done to the St. Louis bridge, it is evident that a wind-pressure of 60 lbs. per square foot was exceeded over a considerable area. This is equivalent to a wind-velocity of about 130 miles per hour, using a coefficient of 0·0035, which, as will be shown later, is a suitable coefficient to use for the flooring which was displaced. As the area of this displaced floor was 180 feet by 18 feet, it is evident that a much higher pressure must have been produced locally by the gusts. In this same tornado a chimney 162 feet high lost its top section, the dimensions of which were 110 feet by 14 feet, and it is stated that a pressure of 85 to 91 lbs. per square foot was needed to effect this destruction. A wind-pressure of this amount corresponds with a velocity of about 180 miles per hour. In another tornado in St. Louis in 1871, a locomotive weighing between 40 and 50 tons was blown over. Calculations show that loaded goods-wagons may, in some cases, need 135 lbs. per square foot to blow them over, and the wind-pressure capable of overturning an engine must have been abnormally high.

In a tornado in the United States on the 19th May, 1879, a 160-foot span bridge weighing 120 tons was blown off its piers without overturning. Such a span might possibly have presented a side area of 1,300 square feet for the two girders; and, assuming a coefficient of friction of 0·45 for the steelwork on its masonry supports, it would have needed a wind-pressure of 83 lbs. per square foot to cause the displacement. This is equivalent to a wind-velocity of 146 miles per hour using a coefficient of 0·0039, which is reasonable for a lattice-girder road-bridge.

It is evident, therefore, that abnormal cases arise in other countries where tornadoes may occur. It is not reasonable to provide against such abnormal cases in a wind-pressure formula for general application in India, although it might be advisable to give the subject special consideration in the cases of important structures in those parts of the country where the storm-conditions are worst, such as the exposed coast section of the Bay of Bengal from Akyab in Burma to Masulipatam and of the Arabian Sea from the Rann of Cutch to Karachi. Other slightly less exposed areas are the coastal strip between Rangoon and Akyab and the Madras coast. From consideration of the foregoing data the Author is of opinion that it would be reasonable to assume a maximum mean wind-velocity in India of 90 miles per hour, rising in the case of gusts to 120 miles per hour.

THE DYNAMIC ACTION OF GUSTS.

It is sometimes stated that a gust should be considered as a suddenly-applied load and that a dynamic increment should therefore be added to the total pressure, to allow for its sudden application. Unfortunately, at present no instrument appears to exist capable of recording the rate of rise of velocity in gusts, and apparently very little consideration has been given to this subject. The Author's own impression is that, although there may be a considerable rise in wind-pressure in a storm in an interval as short as one quarter of a second, yet it takes several seconds for the wind-velocity in gusts to rise from its minimum to its maximum value.

There is very little data for guidance in deciding how the velocity of application of a gust will affect a structure, and what dynamic increment should be applied to the stresses caused by it. Mr. T. Box shows, in his book on the strength of materials,[1] that, if a rolling load be made to traverse a span, there is always a certain velocity at which the load causes the maximum deflection of the span. This maximum deflection is just twice the static deflection under the load. Mr. Box shows also that the speed at which the rolling load would have to travel, in order to produce the maximum deflection, is the speed at which it would cover half the span in the same time as a freely-falling body would require to fall through a vertical height equal to twice the static central deflection of the span under the particular rolling load. He shows that at speeds above or below this particular speed the maximum deflection under the rolling load is always less than at this particular speed. Considering a span such as the Kosi 100-foot deck span, which has a maximum mean deflection of 0·53 inch under the rolling load of one locomotive, it is probable that the increase in deflection due to the maximum wind-pressure on the leeward girder, and to the resultant average increase of stress in the whole girder, would only add about 5 per cent. to this value.[1] If this wind-load were applied at such a rate as to cause the maximum increase of stress in the girder, the deflection might be 10 per cent. of the deflection under the maximum live load, or approximately 0·053 inch. The length of time required for a body to fall freely through a distance of 0·053 inch is about one-sixtieth of a second, and, therefore, for the gust to exert a dynamic increment equal to its static pressure it would have to be applied in one-sixtieth of a second. If the above assumption is true, it is obvious that the dynamic increment due to gusts is practically negligible. Rough calculations show that, if the dynamic increment due to wind-gusts on spans is to be 100 per cent., the gust must be applied in something less than the following periods:—40-foot spans, $\frac{1}{150}$ second; 100-foot spans, $\frac{1}{60}$ second; 150-foot, spans, $\frac{1}{40}$ second; and 300-foot spans, $\frac{1}{20}$ second. It is inconceivable that the rise in velocity in wind-gusts could possibly be anything even approximating to these figures. The conclusion is that, except perhaps in the case of cyclones, the dynamic increment may be omitted from consideration.

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[1] "A Practical Treatise on the Strength of Materials," London, 1883.

A few examples of gusts recorded by the Beckley anemograph at Alipur during cyclones are of interest. In the cyclone of the 21st May, 1902, the wind-velocity rose from zero to 33 miles per hour in 1 minute, and to a maximum of 37 miles per hour within 4 minutes. During a north-wester on the 8th April, 1912, the maximum gusts of 54 miles per hour lasting for 4 minutes were superimposed on an average mean velocity of 40 miles per hour for 12 minutes. During a cyclone on the 28th April, 1914, the wind-velocity during gusts rose from 9 miles to 62 miles per hour in about 2 minutes. Isolated gusts lasted for a few seconds each, and the average wind-velocity for a period of 20 minutes was 30 miles per hour. In the cyclone of the 21st September, 1916, the wind in gusts varied between 6 miles and 45 miles per hour in 5 minutes.

SIZE OF STRUCTURE AS AFFECTING MAXIMUM WIND-PRESSURE.

According to the mean of Dr. Stanton's and Dr. Unwin's experiments the formula connecting pressure with wind-velocity is $P = 0{\cdot}0031\ V^2$. For an average wind-velocity of 90 miles per hour this gives a pressure of 25·2 lbs. per square foot, while for a maximum gust-velocity of 120 miles per hour it gives a pressure of 44·6 lbs. per square foot. Dr. Stanton's experiments show further that there is no variation in the intensity of pressure due to variation in the size of the surface exposed, when the wind is steady. In a variable wind it appears that the size has a very considerable effect on the intensity of pressure. In the experiments made at the Forth Bridge on a plate 20 feet by 15 feet, having an area of 300 square feet, the pressure per square foot recorded averaged only 66 per cent. of that on a plate 1½ foot in diameter. This area of 300 square feet approximates to that of a 50-foot span or to the side of a four-wheeled railway-carriage. It scarcely seems necessary to consider wind-pressures on smaller areas than these. Assuming that the maximum pressure in gusts is developed on a small plate, the maximum pressure per square foot on a large plate approximating to the area of a 40-foot span, or to that of a railway-wagon, might be considered as about 75 per cent. of that on the small square plate. This would give pressures of about 23·3 lbs. per square foot under English conditions and 33·5 lbs. per square foot under Indian conditions.

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[1] In Table VII increase of stress is given in the leeward bottom boom, but this is accompanied by a decrease in stress in the leeward top boom.

Presumably, as the area of a span increases, the effects of gusts are felt less and less, until, on a large square plate approximating to the area of a large span such as 600 to 1,000 feet, the increase of pressure due to gusts would be absolutely negligible; and the assumption could be made that the pressure on these large surfaces was that due to the maximum steady wind-velocity. Such wind-pressure would be 17·5 lbs. per square foot under English conditions and 25·2 lbs. per square foot under Indian conditions. Therefore, for Indian conditions, the pressure, when considered as a pressure on a flat plate, might be reduced from 33·5 lbs. per square foot on an area of the size of a 40-foot span to about 25·2 lbs. per square foot on a large span of 600 feet or more.

EFFECT OF THE SHAPE OF A STRUCTURE ON THE PRESSURE.

In the last section the pressures considered were those assumed to act on square flat plates. But in bridge-work the areas are in the shape of either long bars or lattices. In the case of rectangular areas or bars, Dr. Stanton has shown that the pressure increases with the ratio of length to breadth. If such ratio of length to breadth is 50 to 1, the pressure is 29 per cent. higher than that on a square plate of the same total area. In the case of proportions similar to those pertaining to plate girders the increase would probably be 15 to 20 per cent. Similarly Dr. Stanton has shown that the pressure per square foot on a lattice is considerably higher than that on a rectangle of similar outside dimensions. He shows in the case of a lattice girder, 29 feet by

3 feet 7½ inches, that the pressure per square foot of surface seen in elevation is 26 per cent. higher than that on a rectangle of similar shape and area, or 46 per cent. higher than the pressure per square foot on a square plate of similar total area. In the case of plate girders the depression or cup-shape presented by the flanges will increase the pressure on them by at least 10 per cent. In the case, therefore, of a small plate-girder span of about 40 feet, the assumption must be made that the pressure per square foot given in the last section of this Paper is increased by about 10 per cent. because of this depressed shape, and by a further 20 per cent. for the elongated shape of the girder, giving a total increase of about 30 per cent., and raising the pressure per square foot to 43·5 lbs.

In the case of spans of the lattice type the increase of pressure is shown by Dr. Stanton to be 46 per cent. With the very large lattices in long spans, such as 600 feet, it is probable that this rule would not hold and that the increase of pressure, instead of being 46 per cent. above that on a flat square plate, would be possibly only 35 per cent. above. The pressure on a 600-foot span, would, therefore, be about 25·2 × 1·35, or 34 lbs. per square foot. From this a reasonable assumption to meet Indian conditions would be to allow pressures somewhat as follows :—40-foot span, 44 lbs. per square foot ; 80-foot, 42 lbs. ; 100-foot, 40 lbs. ; 200-foot, 38 lbs. ; 300-foot, 37 lbs. ; 400-foot, 36 lbs. ; 500-foot, 35 lbs. ; 600-foot and over, 34 lbs. The corresponding figures for English practice would be about 30 lbs. per square foot for small spans, diminishing to about 24 lbs. for spans of 600 feet and over. These figures would be satisfactory for use in all cases except very exposed positions on the sea-coast, which should be considered on their merits.

Dr. J. A. L. Waddell[1] suggests for American practice 35 lbs. per square foot on spans up to 200 feet, diminishing to 30 lbs. on 600-foot spans and 25 lbs. on 1,000-foot spans. These figures appear rather low for those parts of the United States where tornadoes are likely to occur, but they appear rather high for the northern and central portions of the States, and have possibly been given as an average for the whole country.

SHIELDING ACTION DUE TO DOUBLE BARS OR DOUBLE GIRDERS.

Recent experiments by Dr. Stanton and older experiments by Sir Benjamin Baker and others show that one flat plate placed behind another is largely sheltered from the wind, and moreover, that in certain cases the total pressure on two such plates is actually less than the pressure upon a single plate by itself. The shielding action roughly causes the total wind-pressure on the two plates to be the following multiple of the pressure on one plate :—

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[1] "Bridge Engineering" vol. i, p. 150, New York, 1916.

Distance plates are spaced apart in diameters		1·0	1·5	2·0	3·0	4·0	5·0
Total pressure on two plates expressed as a multiple of the pressure on a single plate . .	Stanton	0·75	0·75	1·00	1·25	1·50	1·78
	Baker .	1·00	1·25	1·40	1·60	1·80	..

Plate-girder spans may be considered as a pair of plates one behind the other. As deck plate-girder spans generally have their girders spaced from once to twice their depth apart, the pressure on the two plates, according to the experiments cited, might vary from 0·75 to 1·40 times the pressure on a single girder. It is generally safe to assume the pressure on the two girders to be one and a quarter times the pressure on a single girder.

Experiments show that, if a deck plate is fitted, the resistance offered to the wind is reduced as the two plate girders approach more nearly to the shape of a box. The resistance of a box or square pillar is about 90 per cent. of that of a flat plate. A plate-girder deck span with a solid deck, so far as wind-pressures are concerned, may be considered to offer the same resistance as a single girder.

In the case of plate-girder through spans the main girders are considerably farther apart, and the shielding action will not be so well developed. In the case of a through span with an open deck the total wind-pressure on the span is about one and a half times the pressure on a single girder. If a deck plate is fitted, this resistance is reduced and may be safely taken as one and a quarter times the pressure on a single girder.

With regard to open-web or triangulated spans, Dr. Stanton has shown that, using flat latticed girders with a flat plate between them, the wind-pressure on the leeward lattice girder is only 15 per cent. of that on the windward lattice when the two are spaced one depth apart, or 25 per cent. when spaced two depths apart. The lattices used by Dr. Stanton apparently had apertures equivalent to approximately half the total area bounded by the overall dimensions. In modern bridges the apertures will be 60 to 75 per cent. of the overall area, and the shielding action will not be so pronounced. Further, even if there is a deck-plate, the rail-bearers and cross girders add to the resistance considerably, and it is not safe to take the area exposed to the wind as less than one and a half times the total side elevation of the bridge.

Dr. Stanton has shown that, when no deck-plate is provided, the shielding action is not so great, and that, with lattices spaced one depth apart, the pressure on the leeward lattice will be 28 per cent. of that on the windward one. This may be compared with the 15 per cent. when a deck plate is fitted. In modern bridges with open webs and with open decks, therefore, the total area exposed to the wind should be taken as one and three-quarter times the side elevation. In structures such as trestle towers, etc., the area exposed to the wind may also be taken as one and three-quarter times the total side elevation.

In structures built for a double track it will probably be found advisable to allow for slightly greater areas than for similar single-track bridges.

THE EFFECT OF RAIN WITH WIND.

There seems to be an impression that when the wind is heavily charged with moisture, as in severe storms, the pressure on a flat surface will be appreciably greater than when the wind is not so charged. Assuming that raindrops are 5 millimetres in diameter and that the drops fall with a velocity of 5 to 10 metres per second, even with a rainfall of 4 inches per hour the increase in weight of 1 cubic foot of air due to the rain carried therewith would be less than one-quarter per cent., and the increase of pressure would accordingly be inappreciable.

WIND-PRESSURES ON LOADED SPANS.

It has generally been assumed that a considerable reduction of wind-pressure may be allowed when dealing with a loaded span. The general arguments advanced are: (1) that in wind-storms of maximum intensity a train could not get on to a bridge, and (2) that, if a train did get on to a bridge, it would be travelling at a low speed owing to the storm, that the impact would thus be less, and that, therefore, for some obscure reason, the allowance for wind-pressures should also be less.

It will be well to consider these two arguments in a little detail. It is possible that, in an extraordinary storm and with a heavy cross wind, certain trains consisting of empty wagons or of passenger-stock might be blown over. Investigations show that on an average it takes about the following wind-pressures (Table II, p. 15) to overturn empty and full vehicles.

From the Table it will be seen that it is very unlikely that loaded goods-stock would be in danger at any time, except, perhaps, in the case of the 2-foot 6-inch gauge. Crowded passenger-trains on the broad gauge are safe, on the metre gauge they are generally safe, but

TABLE II.—AVERAGE WIND-PRESSURE NEEDED TO OVERTURN ROLLING STOCK.

Gauge.	Type of Stock.	Empty.	Fully Loaded.
		Lbs. per Sq. Ft.	Lbs. per Sq. Ft.
5′ 6″	Coaching	38–60	45–65
5′ 6″	Goods	36–40	110–125
Metre	Coaching	21½–32½	30–36
Metre	Goods	19–25	65–90
2′ 6″	Coaching	15½–23	19–27½
2′ 6″	Goods	13–25	33–70

NOTE.—Appendix A gives details of some typical stock on different gauges and the pressures needed to overturn them.

on the 2-foot 6-inch gauge they are not safe. It would appear, therefore, that there is nothing in general to prevent loaded goods-stock on all gauges from getting on to a bridge in the worst storm. It is possible for loaded passenger-trains on the broad and metre gauges to get on to bridges. It is further possible for 2-foot 6-inch gauge passenger-trains to be caught on bridges, owing to the very sudden and rapid rise of wind-pressures in cyclones. It is, therefore, quite wrong to argue that trains are not likely to be able to get on to bridges during storms of maximum intensity, owing to their being overturned by the wind before getting there. In one of the worst of the seventeen cases cited of trains blown over by the wind the accident occurred on a bridge.

It is sometimes stated that the wind-pressure itself would cause a train to pull up. This, to a certain extent, is supported by evidence, it having been shown in the Paper on trains blown over by the wind mentioned previously that the trains were in many instances brought to a standstill before being blown over. Considering a typical example, a powerful locomotive capable of hauling a load of 1,000 tons up a gradient of 1 in 100 at a speed of 30 miles per hour would be able to haul thirty fully-loaded wagons of about 32 tons each and a brake-van, the weight per linear foot of such a train being approximately 1·4 ton. At low speeds the train-resistance might be 8 lbs. per ton and a draw-bar pull of 3·6 tons would be required to overcome it. At a speed of 30 miles per hour, the train-resistance might have risen to about 20 lbs. per ton, requiring a draw-bar pull of about 9 tons. The resistance due to a gradient of 1 in 100 would be 10 tons. The locomotive would therefore have to be capable of exerting a draw-bar pull of 19 tons. While travelling on the flat at 30 miles per hour, it would be using only about half its full power. Assume that a wind blowing at 90 miles per hour be encountered. Such a wind produces a pressure of about 22 lbs. per square foot on the side of an object shaped like a wagon, and it would exert a side pressure on the train of about the following amount:—

$$P \times L \times H = 22 \times 716 \times 11 = 173{,}000 \text{ lbs.} = 77 \cdot 4 \text{ tons},$$

where P denotes the pressure per square foot = 22 lbs.,

L „ length of train $\frac{1{,}000}{1 \cdot 4} = 716$ feet,

H „ vertical side height of train = 11 feet.

The extra flange-resistance due to the train being pressed against the leeward rail with a pressure of 77·4 tons would only be about $77 \cdot 4 \times \frac{6}{2240} = 0 \cdot 21$ ton. This would barely affect the speed.

If the wind of 90 miles per hour were exactly at right angles to the direction of travel of a train which was itself moving at 30 miles per hour, it would appear, relatively to the train, to be blowing from about 15 to 16 degrees in front of the beam. With a wind blowing from this direction the actual side pressure on the train would not be lessened, while there would be a frictional resistance, due to the component of the wind-pressure acting along the sides of the vehicles, equal to nearly 0·25 of the total side-pressure. The retarding force would be 19·4 tons while the train was travelling at this speed and would tend rapidly to pull it up. This retarding force would fall off rapidly, however, as the speed of the train decreased.

Dr. Unwin has given the following Table showing the relationship between the angle a which the wind makes with a surface, the pressure N normal to the surface, the pressure V on the surface in a direction perpendicular to the wind, and the pressure H on the surface acting parallel with the wind.

a	5°	10°	20°	26½°	30°	40°	50°	60°	70°	80°	90°
N	0·125	0·24	0·45	0·59	0·66	0·83	0·95	1·0	1·02	1·01	1·00
V	0·122	0·23	0·42	0·53	0·57	0·64	0·61	0·50	0·35	0·17	0·0
H	0·01	0·04	0·15	0·27	0·33	0·53	0·73	0·85	0·96	0·99	1·0

From an examination of the Table of various pressures due to the wind at different angles, and from an inspection of the plotted diagram showing them graphically (*Fig. 1*), it will be seen that the wind can vary 30 degrees from the normal to a surface without appreciably affecting the side pressure on that surface.

From the diagram it will also be seen that the maximum retarding force of the wind on a train occurs when the relative direction to the train is about 40 degrees in front of the beam. With the wind at this angle, the total side pressure on the train will be 95 per cent. of its maximum value, or about 73½ tons in the case considered, while the retarding force will be about $77 \cdot 4 \times 0 \cdot 45 = 35$ tons.

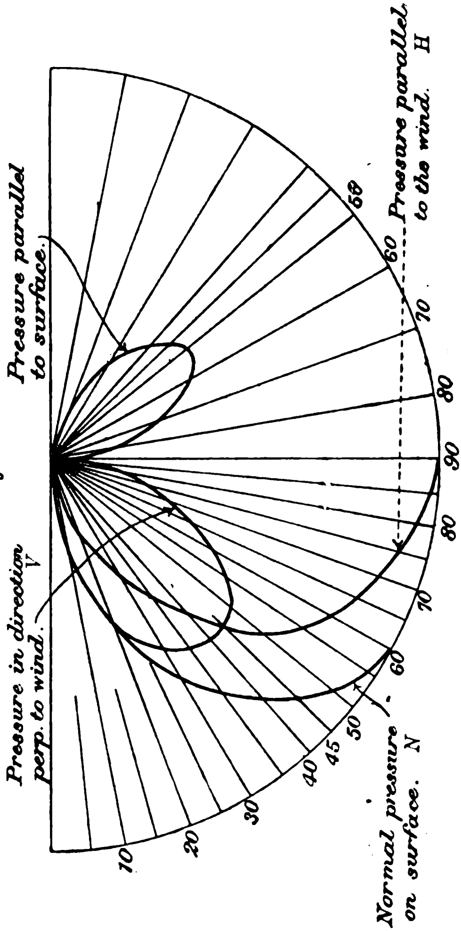

Fig. 1.

When a wind is on the beam or before the beam, a locomotive with a full load behind it is likely to be pulled up. If the load were only half the full load, the locomotive should be able to maintain the maximum speed of 30 miles per hour so long as the wind was on the beam only and did not veer round to some point in front of the beam. If, however, the wind veered to 40 degrees before the beam, the locomotive, even with half a load, would be pulled up. It is quite as likely that the wind will be behind the beam as that it will be in front of the beam. If a wind 90 miles per hour is about 15 degrees behind the beam, and if the train is travelling at 30 miles per hour, there will be very little retardation due to wind-pressure, and only a slightly extra flange-resistance of about 0·2 ton will be caused. With a wind blowing from a direction more than 15 degrees behind the beam there will be no retarding force on the train, but actually an accelerating one. A wind is as likely to blow from one direction as from another. In the conditions assumed, therefore, the chances are about five to seven that the train will not be retarded by the wind. Only a few trains run with the maximum load possible behind the tender; quite a number run with three-quarters or one-half of full load; it is, therefore, probable that in many cases the crossing of an exposed bridge by a train at high speed in the heaviest storm is by no means unlikely.

It is probable that an engine-driver will purposely reduce the speed during a very heavy storm, partly because of the difficulty in seeing, owing either to thick dust or heavy rain, and partly owing to the general feeling of unrest, insecurity, and apprehension felt in a storm of the magnitude considered. With the steam reduced, a train is more likely to be pulled up by sudden gusts.

In a rule specifying the allowance to be made for wind-pressure a margin must be left for bad cases so that, although it may be conceded that the speed of a train is likely to be lower in heavy storms, it can neither be assumed that the speed will be reduced until it is quite low, nor that the train will be pulled up or will only creep across bridges. It might be reasonable to allow that the maximum speed in a storm should be, in general, 5 to 15 miles per hour less than the maximum speed under ordinary conditions. A reduction in the speed of a train cannot affect in any way either the velocity with which the wind is blowing or the pressure produced by such velocity. It may reduce the impact of the live load of the bridge. It may be well to consider the probable effect of such reduction of speed on the impact.

In an investigation recently made on this subject,[1] the critical speeds for the 5-foot 6-inch and 2-foot 6-inch gauges were found to be approximately as given in Table III.

Assuming that the average maximum speed of trains will be reduced in heavy storms by 10 to 15 miles per hour, it is still possible that critical speeds will be attained on spans of 150 feet and more on the broad gauge, and, possibly, on spans of 100 feet and more on narrow gauges. It is believed that on spans of these sizes the impact depends more on critical speeds being attained than on any other factor. It would, therefore, appear that on such spans, even with the reduced maximum speed suggested as being probable in heavy storms, there is no certainty that the impact will be reduced. It

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[1] See Author's Paper on "Impact on Bridges." Technical Paper No. 199, published by the Technical Section of the Indian Railway Board, Simla, April, 1919.

TABLE III.—CRITICAL SPEED: MILES PER HOUR.

Span: Feet	40	50	60	80	100	125	150	200	Probable Maximum Speed.
5′ 6″ gauge.									
Goods-engine, 4′ 8″ driving-wheels	60[1]	54[1]	49[1]	42	35–43	34	31	26	35–45
Passenger-engine, 6′ 1½″ driving-wheels	82[1]	73[1]	67[1]	59	50–61	49	45	36	50–60
2′ 6″ gauge									
Goods-engine, 2′ 10″ driving-wheels	46[1]	39[1]	32	28	23	..	..	..	25–30
Passenger-engine, 3′ 6″ driving-wheels	60[1]	50[1]	42	37	31	..	..	..	35–45

[1] Critical speeds not likely to be reached on these spans.

may be conceded that in wind-storms of maximum intensity the impact on the smaller spans may be reduced, owing to the probable reduction in speed. In the present state of knowledge regarding impact it would scarcely be advisable to specify any very substantial reduction in the coefficients to be used when both maximum live load and wind loads are considered. It would be preferable to allow a considerable increase in the unit working-stresses when such adverse conditions are considered conjointly.

There are two logical reasons for specifying a reduced wind-pressure on a loaded span:—

(*a*) Because the total area exposed to the wind is increased, and hence the allowance necessary for the rise in the pressure due to gusts above that corresponding with the maximum mean wind-velocity is not so great as is necessary with a smaller area.

(*b*) Because the resistance offered by a body like a train is not so high as that offered by a surface like a bridge.

Table IV gives the approximate exposed areas of girders and trains for the 5-foot 6-inch gauge. It also gives reasonable pressures to be allowed on girders and on train-surfaces, and finally the average pressure which might be allowed on the combined surface of bridge and train. When fixing reasonable allowances for wind-pressures on train-surfaces, consideration has been given to the experimentally-verified fact that a closed box or like structure offers to the wind a resistance only 90 per cent. of that of a flat plate of the same area as the side elevation of the box.

The suggested wind-pressures for 5-foot 6-inch gauge structures

TABLE IV.

Span. Length in Feet.	Plate. 40	Plate. 60	Plate. 80	Plate. 100	Open Web. 100	Deck. 150	Through. 150	Through. 170	Deck. 200	Deck. 300
Girder area . . . . . sq. ft.	262	588	1,000	1,280	1,290	1,650	1,730[1] 2,100[2]	1,230[1] 1,800[2]	2,600	..
Train area . . . . . sq. ft.	440	650	865	1,070	1,070	1,580	1,580	1,800	2,120	..
Pressure on girder . { lbs. per sq. ft. }	44	43	42	40	40	39	39	38	38	37
Pressure on train-surface . . . . . { lbs. per sq. ft. }	30	29	28¾	28½	28½	27¾	27¾	27½	27	25
Total pressure on girder lbs.	1,150	2,510	4,200	5,120	5,168	6,450	6,760	4,680	9,900	..
Total pressure on train lbs.	1,320	1,890	2,490	3,050	3,050	4,390	4,390	4,950	5,930	..
Total pressure on loaded structure . . . . . . lbs.	2,470	4,430	6,690	8,170	8,170	10,840	11,150	9,630	15,830	..
Total area of loaded structure sq. ft.	702	1,238	1,865	2,350	2,360	3,230	3,310	3,030	4,720	..
Average pressure on loaded structure[3] { lbs. per sq. ft. }	35	35¾	35¾	34¾	34½	33½	33¾	31¾	33½	..

[1] Part of loaded girder area unshielded by train.

[2] Unloaded span.

[3] There should be a slight further reduction of possibly ¼ lb. to ½ lb. in pressure on loaded span owing to increased area exposed.

under Indian conditions are, therefore, as in the following Table:—

TABLE V.

Span (length in feet)		40	60	80	100	150	200	300	400	600	1,000
Pressure: Lbs. per sq. ft.	Unloaded	44	43	42	40	39	38	37	35½	34	33
	Loaded	36	35	35	34½	33½	33	32½	32	31½	31

It seems that, as a rough rule, it is safe to specify, for the wind-pressure on loaded 5-foot 6-inch gauge spans, 5 lbs. per square foot less than for unloaded spans. For small spans the reduction could be really a little more than 5 lbs., but for large spans it should be less than this amount; so that, for a span like the Quebec bridge, the difference in wind-pressure per square foot on the loaded and unloaded structures would be negligible. Dr. Waddell specifies[1] that the wind-pressure per square foot on the loaded structure may be taken as 5 lbs. per square foot less than that on the unloaded structure.

Table VI gives corresponding figures for the 2-foot 6-inch gauge with a view to some conclusion being drawn as to whether any reduction in wind-pressures is really allowable on small gauges:—

TABLE VI.

Span: Feet	40	50	60	80
Girder area . . . . . sq. ft.	250	360	460	925
Train area . . . . . sq. ft.	359	440	520	690
Total area . . . . . sq. ft.	600	800	980	1,615
Pressure on girder . lbs. per sq. ft.	44	44	43	42
Pressure on train . lbs. per sq. ft.	31	30	29	29
Total pressure on girder. . . lbs.	1,100	1,590	1,980	3,880
,, ,, on train . . . lbs.	1,110	1,320	1,510	2,000
,, ,, on loaded structure lbs.	2,210	2,910	3,490	5,880
Average pressure on loaded structure lbs. per sq. ft.	37	36½	35	36½

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[1] "Bridge Engineering," vol. i, p. 150.

The wind-pressures for loaded narrow gauge spans should not vary substantially from those specified for the broad gauge.

The logical way to treat wind-pressures is to specify the maximum pressures which are at all likely to occur, and to allow for these in all cases in designing bracings. Owing, however, to the fact that maximum wind stress and maximum impact are very unlikely to occur simultaneously with maximum loading of a structure, quite a considerable rise in the unit working-stresses might be allowed above those usually specified when all these maxima are computed and their sum is taken. Under ordinary conditions the primary, or working unit stresses specified amount to 55 to 60 per cent. of the elastic limit of the material. To these usually computed primary stresses as much as 33 per cent. may be added for secondary stresses —which latter will account for 20 per cent. of the range of stress within the elastic limit, leaving about 20 to 25 per cent. for indefinite stresses such as those due to temperature, erection, uncertainty of action due to all forces not meeting at a point, etc. It would probably be safe to allow a rise above the ordinary unit working-stress of 25 to 33 per cent. A rise of 25 per cent. would still leave a margin of about 10 per cent. of the full range within the elastic limit for these indefinite stresses, provided that high secondary stresses had been avoided.

METHODS OF CALCULATING WIND STRESSES IN BRIDGES AND THE APPROXIMATE RANGE OF STRESS DUE TO THE WIND

There can be not the slightest doubt that the overturning action of the wind on both the bridge and the train should be calculated and allowed for. The overturning-moment on the train causes an excess of pressure on the leeward rail and therefore increases the total load carried by the leeward girder, while relieving the windward girder by the like amount. The overturning action on the bridge itself may either act in the same manner or be in antagonism to this action, depending altogether upon the position of the bearings in relation to the centre of pressure of the wind on the structure. It will generally be found on examining the structure that the stresses due to the overturning-moment are considerably more important than the stresses due to the horizontal wind-pressure on the structure and train. In many cases the overturning-moment stresses quite dwarf those due to the horizontal force alone. The critical section of a span will generally be found to be the lower boom of the leeward girder, in which both the wind stresses due to the overturning-moment and those due to the horizontal component act in unison and tend to increase the direct stresses

TABLE VII.—WIND STRESSES.

5-foot 6-inch Gauge Leeward Girder, Bottom Boom.

Wind stresses stated as percentages of total dead, live, and impact stresses.

Span. Length in Feet.	Plate Deck. 40	Plate Deck. 60	Plate Deck. 80	Plate Deck. 100	Deck, Open Web. 100 Kosi.	New Deck, Open Web. 100	Deck. 150	Through New. 150	Through. 170	Deck. 200
Horizontal component of wind	..	2·5	3·5	2·9	5·0	6½–9½	6½–11¼	14–22	..	3·5
Overturning-moment . .	12·7	14·7	18·3	23·0	7·0	8½	7¼	8	..	3·0
Total wind stress . . .	..	17·2	21·8	25·9	12·0	15–18	14–18½	22–30	..	6·5
Strength of span . . . (Standard)	B + 25%	B + 25%	B + 25%	B + 25%	B	B + 25%	B	B + 25%	..	B + 25%
Spacing of main girders .	5′ 9″	7′ 0″	7′ 0″	7′ 3″	10′ 0″	10′ 0″	13′ 2″	16′ 5″	16′ 5″	17′ 0″

5-foot 6-inch Gauge Windward Girder, Top Boom.

Span. Length in Feet.	Deck, Open Web. 100	New Deck, Open Web. 100	Deck. 150	Leeward Through New. 150	Through. 170	Deck. 200
Horizontal component of wind	9¾	20¾	12½	−1¾–2¾	..	11·7
Overturning-moment	−6¾	−10¼	−7¼	9	..	−3·0
Total wind stresses	3	10½	5¼	6¼–7¼	..	8·7

due to the ordinary dead, live, and impact loads. Table VII gives details of the stresses in certain typical 5-foot 6-inch gauge spans due to wind. These stresses have been stated as percentage increases on the total direct stress due to the combined dead, live, and full impact loads.

It will be seen that there is a very great variation in the wind stresses. In these spans the percentage increase of stress due to the wind ranges from 6½ to 30 per cent. of the total direct stresses. The 30-per cent. increase occurs in a 150-foot through span, and it is therefore obvious that in the cases of such spans the wind stresses should always be computed and, if necessary, allowed for: the general practice of considering off-hand that the wind stresses in ordinary spans must necessarily be within the 25 per cent. increase permitted is not by any means a safe assumption. Another point which is brought out by this Table is that the stresses due to the overturning-moment are generally more important—sometimes as much as eight times as important—as those due to the horizontal component of the wind. In general it will be found that large deck spans of 150 feet and more are not severely stressed by the wind owing to the "sail" areas above and below the bearings tending to balance each other.

Large through spans need careful consideration. Particular attention should be paid to the end diagonals in such spans, which not only have to carry heavy direct compression, but are liable also to a twisting action owing to their having to transfer the whole of the wind loads on the top booms down to the bearings.

Consideration of the action of the wind is particularly necessary in the case of narrow-gauge bridges, because the main girders of the smaller spans up to about 100 feet are generally much closer together than those of broad-gauge bridges of like span. This close spacing of the main girders very much reduces the strength of the spans to resist either horizontal forces or overturning forces. In the design of 2-foot 6-inch gauge spans, wind stresses have not

TABLE VIII.—COMPARATIVE WIND STRESSES IN LEEWARD BOTTOM FLANGE.

Span. Length in feet.	Deck. 40	Deck. 50	Deck. 60	Deck. 80
Horizontal component of wind . .	3·0	4·0	5·25	6·75
Overturning-moment	27·2	30·0	34·0	37·0
Total wind stress	30·2	34·0	39·25	43·75
Girder spacing.	3′ 9″	3′ 9″	3′ 9″	4′ 3″

been given sufficient consideration. Table VIII shows that, when such wind stresses are correctly computed, and when the overturning-moments are allowed for, as they always should be, these stresses are very much higher than is commonly realized.

It will be seen that the overturning-moment may be as much as eight times as important as the horizontal component of the wind in increasing the stresses in the critical section, which is the leeward bottom flange. It will also be seen that the stresses run very high indeed in 2-foot 6-inch gauge deck plate-girders. Even in the case of the 40-foot span the wind load may increase the total direct stresses due to dead, live, and full impact loads by 30 per cent.; and thus under the present bridge rules an increase of section in the flanges may be required. In the case of the 80-foot span, the wind load increases the total direct stresses by nearly 44 per cent.

Appendix B gives the calculations of the wind stresses in a 5-foot 6-inch gauge 150-foot through span, and Appendix C gives those for a 2-foot 6-inch gauge 60-foot deck plate-girder span.

The Paper is accompanied by fourteen prints, from which the Figures in the text and in the Appendixes have been prepared.

APPENDIXES.

APPENDIX A.—Dimensions and Weights of, and Wind-Pressures needed to Overturn, Typical Rolling Stock.

	Overall Length.		Side Area.				Tare Weight.	Gross Weight.	Heights above Rail of Lower and Upper Edges, excluding Wheels.				Wind-Pressure needed to Overturn Empty Vehicle.
5-Foot 6-Inch Gauge.	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Tons.	Tons.	Ft.	Ins.	Ft.	Ins.	Lbs. per Sq. Ft.
Coaching.													
M. & S. M. :—													
1st and 2nd class, bogie . . .	70	2	67	6 ×	9	9	44·1	..	3	3	12	10	54·8
3rd class, bogie .	64	2	61	0 ×	9	6	38·3	..	3	3	12	9	53·0
3rd class, 4-wheel	31	2	28	0 ×	10	0	13·25	..	2	11	12	11	38·0
N.G.S.R. :—													
3rd class, bogie .	64	2	60	2 ×	9	9	28·7	..	3	3	13	0	38·7
3rd class, 4-wheel	31	2	27	6 ×	9	9	11·65	..	3	3	13	0	34·2
Goods.													
N.G.S.R. :—													
Covered goods A	21	11	19	9 ×	9	6	7·75	24·23	2	6	12	3	36·0
E.I.R. :—													
Covered goods A	25	8	23	0 ×	9	3	8·75	27·75	3	3	12	6	33·3
Metre Gauge.													
Couching.													
Burma R. :—													
bogie	44	0½	40	0 ×	9	0	13·16	19·66	1	10	10	10	22·25
brake	49	0½	45	0 ×	9	0	16·6	22·47	1	10	10	10	25·0
4-wheel, 3rd class	23	9	21	8 ×	8	10	5·8	8·17	1	6	10	4	19·5
4-wheel, 1st and 2nd class . .	23	9	21	8 ×	8	7	6·7	7·70	1	6	10	1	23·8
bogie, 1st class .	50	4½	47	8 ×	9	4	22·1	23·5	1	10	11	2	29·4
N.G.S.R. :—													
bogie, 2nd and 3rd class . .	57	6	54	8 ×	9	0	19·55	..	2	2	11	2	23·0
Goods.													
N.G.S.R. :—													
Covered goods .	21	10	18	0 ×	8	6	4·69	15·91	2	2	10	8	17·5
A.B.R. :—													
Covered goods .	18	8	15	0 ×	7	8	3·89	13·89	2	0	9	8	22·3
	21	8	18	0 ×	8	0	4·36	13·91	2	5	10	5	18·1
,, ,, bogie	39	10	36	0 ×	8	2	10·62	28·03	2	4	10	6	21·7
2-Foot 6-Inch Gauge.													
Coaching.													
Bogie, coupé . .	31	4½	28	2 ×	8	2	7·3	..	1	11	10	1	15·7
4-wheel, 2nd class .	17	10	15	2 ×	7	8	3·0	..	1	8½	9	4½	14·2
Bogie, 3rd class .	26	4	23	2 ×	7	7	4·7	..	1	5½	9	0½	15·4
K.S., bogie, 1st class	33	0	31	0 ×	8	0	9·3	10·9	1	4	9	4	21·0
,, ,, 3rd ,,	33	0	31	0 ×	8	2	6·86	10·25	1	2	9	4	15·4
Goods.													
Covered goods .	18	9	16	3 ×	7	2	2·5	6·5	1	10	9	0	11·9
do. bogie . . .	22	4	20	0 ×	7	2	4·0	9·5	1	10	9	0	15·35
K.S., do. 4-wheel .	17	0	14	0 ×	7	6	4·0	11·5	1	6	9	0	21·8
,, do. bogie . .	30	6	27	6 ×	8	0	7·8	25·05	1	10	9	10	18·1
	33	0	30	0 ×	7	4	7·5	25·6	1	6	8	10	16·0

APPENDIX B.

Wind Stresses on 150-Foot Through Span.

The side area of the unloaded span may be taken as

Deck system :—158 feet × 2·75 feet	435
Bottom boom :—158 feet × 1·25 feet	198
Verticals :—18·125 feet × 1·11 × 3	60
18·125 feet × 1·2 × 4	87
Diagonals :—25·66 feet × 1·35 × 2	69
25·66 feet × 1·25 × 2	64
25·66 feet × 1·0 × 4	103
Top boom :—119 × 1·35 feet	161
	1,177 square feet

say, 1,200 allowing for gusset plates.

Taking one-and-three-quarter times this area as the exposed wind area, wind-pressure on unloaded span $\frac{2,100 \times 2\cdot5}{100} = 52\frac{1}{2}$ tons. Of this 16·0 tons acts on the top boom and 36·5 tons on the bottom boom.

Taking the loaded span, the following are the exposed areas—allowing one-and-three-quarter times the horizontal projection of the ironwork, unshielded by train, plus full train area :—

Deck system	762
Bottom boom	347
Verticals	116
Diagonals	185
Top boom	282
Gussets	40
Train 158 × 10 feet	1,580
	3,312 square feet.

The total wind-pressure on loaded structure $\frac{3,312 \times 1\cdot5}{100} = 49\cdot6$ tons. Of this the approximate amount which acts on the top boom system is 9·2 tons, while the remainder, or 40·4 tons, acts on the bottom boom. When considering maximum stresses on the span, it must be assumed to be loaded The unloaded structure must be taken, however, to consider the stability of the span against overturning.

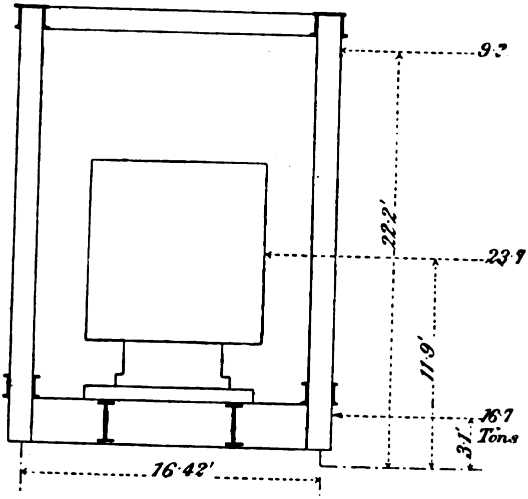
Wind Stresses in the Girders.—The plane about which the span may be taken as tending to overturn is that through the bearing pin, i.e., 10 inches below the sole plate of the girder This point is 4 feet 3¾ inches below the centre line of the bottom boom, or 4 feet 11¼ inches below rail-level.

Considering the case of the loaded span, the total horizontal pressure is 49·6 tons. It may be divided as shown in *Fig. 2,* between the top boom, train, and bottom, so far as overturning is concerned. The total overturning-moment on the bridge is $(9 \cdot 2 \times 22 \cdot 2) + (23 \cdot 7 \times 11 \cdot 9) + (16 \cdot 7 \times 3 \cdot 1) = 204 \cdot 5 + 282 \cdot 2 + 52 \cdot 0 = 538 \cdot 7$ foot-tons.

This overturning-moment may be considered as balanced by an upward force of $\frac{538 \cdot 7}{16 \cdot 42} = 32 \cdot 7$ tons at the leeward bearings and by an equal opposite force on the windward bearings.

Stresses Due to Horizontal Component of the Wind-Pressure.—It is assumed, in calculating wind stresses in the loaded span, that only 9·2 tons is carried by the top boom and that the balance, 40·4 tons, is carried by the lower system—the deck in this case.

Fig. 2.

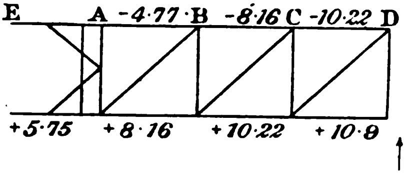


The wind stresses in top booms due to the horizontal component of the wind cause a load per bay of $\frac{9 \cdot 2}{6} = 1 \cdot 53$ ton, or, if the end diagonals are considered as the outer bays of the system, $\frac{9 \cdot 2}{8} = 1 \cdot 15$ ton.

The coefficients for the stresses in the booms (*Fig. 3*) will be :—bay EA, 28 ; AB, 48 ; BC, 60 ; and CD, 64. The length of the bay is 19·5 feet, and centre to centre of booms = 16·42 feet.

The actual boom stresses will be :—EA, total stress, $4 \cdot 77 \times 1 \cdot 414 = 5 \cdot 75$ tons per square inch ; AB, 8·16 ; BC, 10·22 ; CD, 10·9.

Fig. 3.

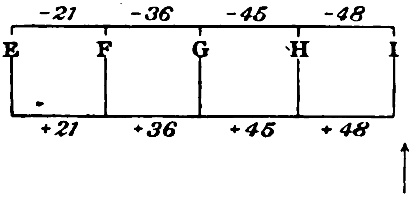


The wind stresses in the bottom booms due to the horizontal component of the wind are given by :—Load per bay $\frac{40 \cdot 4}{8} = 5 \cdot 05$ tons ; stresses in tons : bay EF 21·0 ; FG, 36·0 ; GH, 45·0 ; HI, 48·0 (*Fig. 4*).

Stresses Due to the Overturning-Moment of the Wind.—The overturning-moment of the wind has the same effect as if the windward girder were lightened by 32·7 tons and the leeward girder loaded with a similar excess weight. Three-quarters of this reduction or excess in weight may be taken as a moving wind load and the other quarter as a stationary load.

The total load per bay is $\frac{32 \cdot 7}{8} = 4 \cdot 1$ tons, of which 3·1 tons is moving and 1·0 ton stationary.

Fig. 4.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E
F
G
H
I
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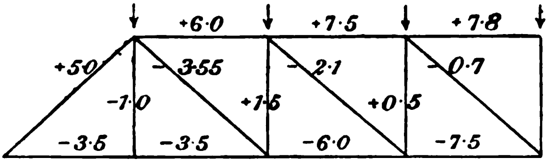
+36
+45
+48

Fig. 5.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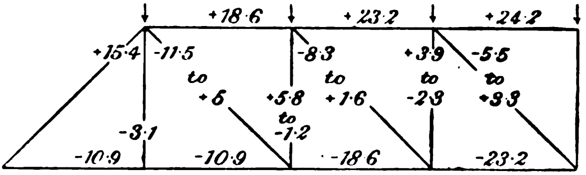
-3·5
-6·0
-7·5

Fig. 6.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+15·4
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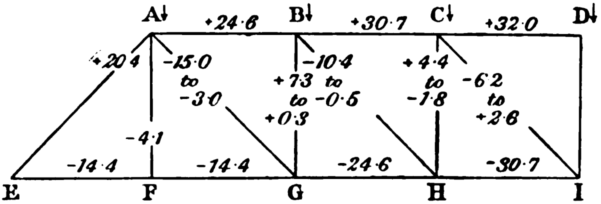
-8·3
+5·8 to -1·2
to +1·6
+3·9 to -2·3
-5·5 to +3·3
-3·1
-10·9
-10·9
-18·6
-23·2

Fig. 7.


A
B
C
D
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+30·7
+32·0
+20·4
-15·0 to -3·0
-10·4
+7·3 to +0·3
to -0·5
+4·4 to -1·8
-6·2 to +2·6
-4·1
-14·4
-14·4
-24·6
-30·7
E
F
G
H
I

The above figures show the wind-loading stress-distribution.

The stresses in the leeward girder due to a stationary load of 1·0 ton per bay applied at each lower apex (*Fig. 5*).

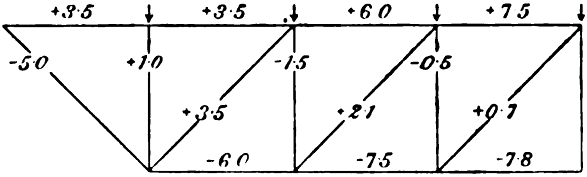
The stresses due to the moving part of the load of 3·1 tons per bay (*Fig. 6*).

The total overturning-moment wind stresses in the leeward girder (*Fig. 7*).

Stresses in Windward Girder Due to Overturning-Moment.—Stresses due to a stationary wind load of 1·0 ton per bay (*Fig. 8*).

Stresses due to a moving wind load of 3·1 tons at each apex (*Fig. 9*).

Fig. 8.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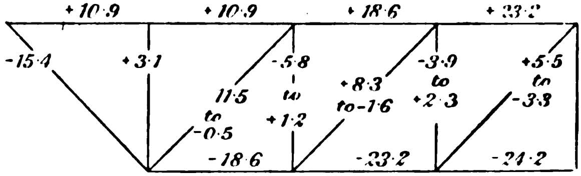
+0·7
-6·0
-7·5
-7·8

Fig. 9.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+23·2
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+3·1
-5·8
to
+1·2
11·5
to
-0·5
+8·3
to-1·6
-3·9
to
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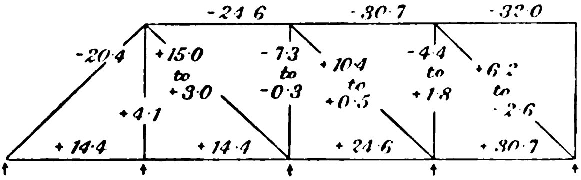
+5·5
to
-3·8
-18·6
-23·2
-24·2

Fig. 10.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to
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to
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to
+0·5
-4·4
to
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to
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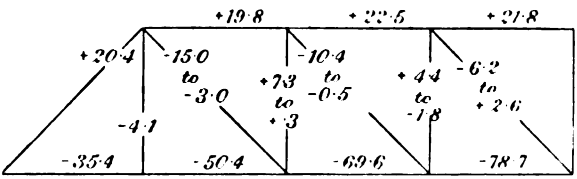
+4·1
+14·4
+14·4
+24·6
+30·7

Fig. 11.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+21·8
+20·4
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to
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-10·4
to
-0·5
+7·3
to
+·3
+4·4
to
-1·8
-6·2
to
+2·6
-4·1
-35·4
-50·4
-69·6
-78·7

Total overturning-moment wind stresses in windward girder (*Fig. 10*).

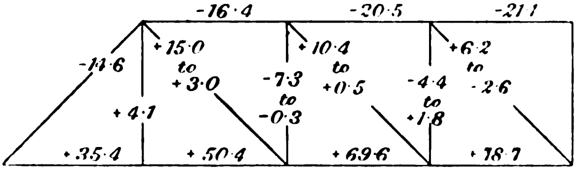
Combined wind stresses due to horizontal side-pressure and overturning-moment of the wind combined, leeward girder (*Fig. 11*). This is obtained by combining the results shown in *Figs. 3, 4* and *7*.

Similar results for the windward girder are shown in *Fig. 12*, being obtained by combining *Figs. 3, 4* and *10*.

Fibre stress, tons per square inch, due to combined horizontal pressure and overturning-moment of the wind, leeward girder (*Fig. 13*).

The wind stress in the bottom boom of the leeward girder may rise to 2·47 tons per square inch, or 30. per cent. of the amount permissible for direct stresses. As only an excess of 25 per cent. is allowed by the bridge rules, some increase of section might be needed to take care of the excess. In this particular case the deck system helps out the bottom booms, and no increase was considered necessary. If only the horizontal component of the wind-pressure had been considered, the increase of stress in the bottom boom of the leeward girders would have appeared to be 1·76 ton per square inch, or 22 per cent. of the allowable direct stresses.

Fig. 12.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to
+0·5
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to
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to
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+35·4
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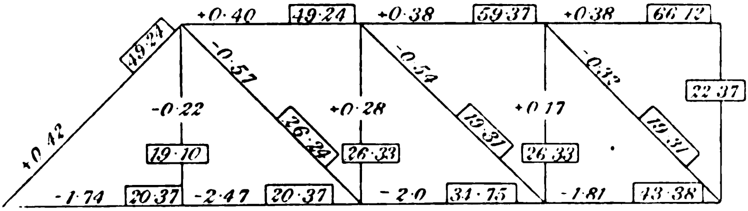
+69·6
+78·7

Fig. 13.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-0·22
19·10
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+0·28
26·33
-0·54
19·31
+0·17
26·33
-0·32
19·31
22·37
-1·74
20·37
-2·47
20·37
-2·0
34·75
-1·81
43·38

APPENDIX C.

Wind Stresses on 60-Foot Span.

2-foot 6-inch gauge.

Data.

Girder 65 feet overall length, 5-foot deep webs.
Add for flange plates, sleepers, and rails, 7½ inches.
Train surface 65 feet by 8 feet.
Wind-pressure on loaded span 1·5 ton per 100 square feet.

Total Wind Load on Span.

$$\text{On girders } 65 \times 5{\cdot}625 \times \frac{1{\cdot}5}{100} \times 1{\cdot}25 = 6{\cdot}88 \text{ tons}$$

$$\text{On train } 65 \times 8 \times \frac{1{\cdot}5}{100} = 7{\cdot}8 \text{ tons}$$

$$\overline{14{\cdot}68 \text{ tons.}}$$

Case 1.

Considering the horizontal component of wind only, the horizontal bending-moment on the span is $14{\cdot}68 \times \frac{61{\cdot}66}{8} \times 12 = 1{,}365$ inch-tons. It may be considered that two-thirds of this bending-moment is resisted by the top flanges and one-third by the bottom flanges.

The net section of the bottom flange is 18·56 square inches.

One-half the gross section of the web is $60 \times \frac{3}{8} \times \frac{1}{2} = 11{\cdot}25$ square inches, making a total area of 29·81 square inches.

The effective depth—distance between centres of girder—is 45 inches.

The wind stress on the lower flange $= \frac{1{,}365}{3 \times 29{\cdot}81 \times 45} = 0{\cdot}338$ ton per square inch.

If the stresses are considered as being resisted by the flange-section only, this stress rises to $\frac{1{,}365}{3 \times 18{\cdot}56 \times 45} = 0{\cdot}544$ ton per square inch.

The maximum flange stress on tension flange under live loads is 6·51 tons per square inch.

Adding wind stresses the total becomes 6·848 tons per square inch or 7·054 tons per square inch in each case respectively.

In the case of the compression flange the gross sectional area of the flange is

Angles	7·498
Plates	15·0
	22·498 square inches.

One-half the gross section of the web $60 \times \frac{3}{8} \times \frac{1}{2} = 11{\cdot}25$ square inches.

The flange stress, considering the flange section + ½ web, is $\frac{1{,}365 \times 2}{3 \times 33{\cdot}75 \times 45} = 0{\cdot}597$ ton per square inch; but considering the flange only it becomes $\frac{1{,}365}{3 \times 22{\cdot}5 \times 45} = 0{\cdot}9$ ton per square inch.

The maximum fibre stress in the compression flange under the rolling load is $\frac{8{,}305 \times 30{\cdot}4}{40{,}325} = 6{\cdot}26$ tons per square inch.

The total maximum flange stress on the compression side under dead load, live load with impact, and wind loads combined is $6{\cdot}26 + 0{\cdot}597 = 6{\cdot}857$ tons per square inch when one-half of the web helps to resist wind stresses. The stress is $6{\cdot}26 + 0{\cdot}9 = 7{\cdot}16$ tons per square inch when only the flange is allowed as resisting wind.

In the case of a plate girder there can be no objection whatever in considering that the web helps the flanges in resisting wind stresses. The stress under all loads combined, including wind, therefore, would be

6·848 tons per square inch in leeward tension flange,
6·857 tons per square inch in windward compression flange.

Case 2.

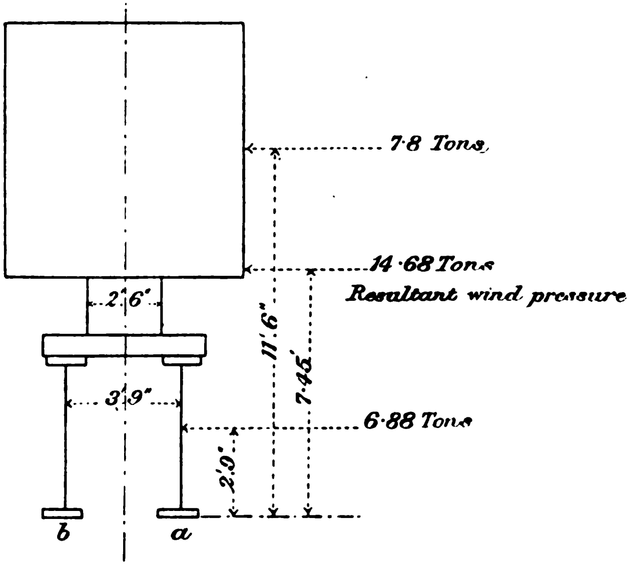
Taking into consideration the overturning-moment of the wind, which tends to cant the train and girder and thus to bring an increased load on to the leeward girder, lessening that on the windward one, the overturning-moment on the span is $7{\cdot}8 \times 11{\cdot}5 + 6{\cdot}88 \times 2{\cdot}75 = 90 + 18{\cdot}85 = 108{\cdot}85$ foot-tons (*Fig. 14*).

This is balanced by a downward force of 29·15 tons at *a*, an upward force of the same amount, 29·15 tons, at *b*, and a frictional resistance of 14·68 tons acting horizontally on the bearing plates. It is evident, therefore, in considering the overturning-moment of the wind, that it is the same as bringing a load of 29·15 tons upon the leeward girder and reducing the load on the windward one by the same moment. The bending-moment due to the extra vertical load of 29·15 tons on the leeward girder is $\frac{29\cdot 15 \times 61\cdot 66 \times 12}{8} = 2{,}690$ inch-tons.

The flange stress due to this, by the moment of inertia method, is $\frac{2{,}690 \times 31\cdot 6}{40{,}325}$ $= 2\cdot 2$ tons per square inch in the tension flange, and $\frac{2{,}690 \times 30\cdot 4}{40{,}325} = 2\cdot 02$ tons per square inch in the compression flange.

When the overturning-moment as well as the horizontal wind stresses are taken into account, the stresses in the tension boom of the leeward girder alone need be considered. In this flange both the overturning-moment of the wind and its horizontal component tend to increase the stresses, while in all other flanges these forces counteract each other to a certain extent.

Fig. 14.



On the lower flange of the leeward girder, therefore, the following stresses exist:—

Stresses due to dead load, live load, and impact .	6·51 tons per sq. in.	100 per cent.
Stresses due to horizontal component of wind . .	0·338 ton per sq. in.	5¼ per cent.
Stresses due to overturning-component	2·2 tons per sq. in.	34 per cent.
Total . . .	9·048	139¼

Discussion.

The President. The PRESIDENT, in moving a vote of thanks to the Author, remarked that The Institution was indebted to him for an exceedingly interesting Paper, representing a very large amount of work, and containing numerous points for discussion.

Dr. Stanton. Dr. T. E. STANTON wished to supplement the Paper by making a few remarks on the experiments which had been carried out at the National Physical Laboratory since the date of his last Paper on the subject in 1908.[1] That work had originated with the dissatisfaction which engineers felt with the ordinary wind-pressure formula $P = KV^2$ as a means of calculating the forces experienced by structures in actual winds. One reason for that dissatisfaction was the inaccuracy of the coefficient K. At the time of the Tay bridge disaster it was frequently taken to be as high as 0·005 (a so-called theoretical value), whereas the true value for a square plate was about 0·003. Further, the velocity was much overestimated owing to the fact that for the Robinson cup readings a factor of 3 was taken, instead of the correct value, 2·2. But there remained one other thing, the lateral variation in the wind, in regard to which Sir Benjamin Baker used to say that the wind filaments moved in echelon, and consequently the maximum velocity did not obtain at the same instant at all points of a surface normal to the direction of the wind. The first step at the laboratory had been to arrange experiments in a uniform current of air on scale models of a structure in a wind-channel. As that was, he believed, the first wind-channel ever made, the accuracy of velocity-measurement was not high, and it was found afterwards that the speed of the air in the channel had been underestimated by 3 per cent., which made a difference of 6 per cent. in the formula, so that the real value of K for a square plate was 0·0029, and not 0·0027 as given in the Paper. In other respects he thought the results given were quite reliable, and it was gratifying to him that the Author had quoted them so extensively. There remained the question of size; and that question had to be divided into two. There was first the purely dimensional effect of size—a matter with which aircraft-constructors had to deal—which would exist if the wind were quite uniform. The effect, which did not depend on wind-structure at all, had been the subject of his

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[1] Minutes of Proceedings Inst. C.E., vol. clxxi, p. 175.

Dr. Stanton.

Paper in 1908. Experiments were made in the open on plates up to 100 square feet in area, and the method of reduction of the observations was such that the results were equivalent to those which would be obtained in a uniform current of air. It was found that, as far as engineers were concerned, the dimensional effect could be practically neglected. The value of *K* for a square plate was approximately 0·0031. The other effect of size was the lateral variation, on which they had been working more or less for the last 15 years. That was a very difficult problem: it meant an investigation of the structure of the wind. Some preliminary work was done in the first 2 or 3 years in the grounds of the laboratory; and, after that had been completed, by the kindness of Sir John Wolfe Barry the Bridge House Estates Committee of the City of London allowed the apparatus to be installed on the Tower Bridge. It was impossible to get records very frequently, owing to the scarcity of winds, but some were obtained. During the war the first gun discharged from the Tower Bridge put the apparatus completely out

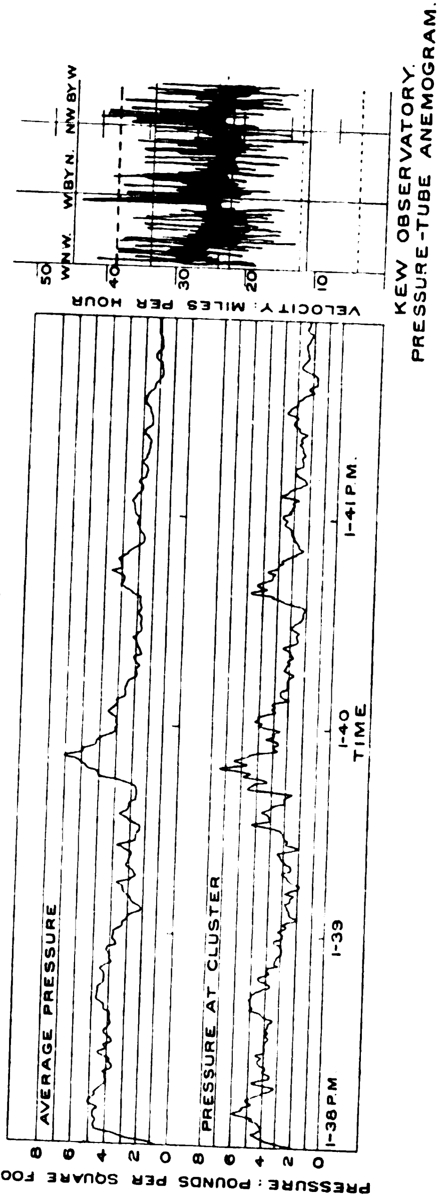

Fig. 15.

Dr. Stanton. of action. It had been set up again, and was now in working order. The apparatus was erected on the upper footways 130 feet above water level. It consisted of two systems of pressure tubes, one system consisting of two clusters of six tubes each at two points, one on the windward side and one on the leeward side, the pressure difference in which was considered to be the pressure at the point at which the cluster was situated. The tubes were connected to a recorder, and a record of the pressure-difference between the two points was taken. The other system was precisely similar, but the tubes were distributed over the area of the girders, so that a record was also obtained of the average pressure-difference between the two sides of the girders. It was possible to derive a factor which enabled these pressures to be expressed in lbs. per square foot on the girder. *Fig. 15* showed a reproduction of one of the records. The lower curve, which gave the variations of pressure at the cluster, was more irregular than the upper, which gave the average; but the important point was that the record showed that the maximum gust which blew during that particular period did produce nearly the same effect on the whole girder as if the pressure had been uniform and equal to its pressure at the point. Therefore the suggestion that the wind-pressure factor could be reduced on account of lateral variation did not hold good—at all events, not in that case. For comparison the figure also showed the simultaneous record of the Dines pressure-recorder at Kew, which indicated that the gusts attained a maximum speed of about 40 miles per hour. There were other records at the Laboratory which seemed to bear out the same conclusion. He was almost sure in his own mind that the structure effect was not a thing engineers ought to take much account of. No doubt it would be objected that these records referred to winds of about 40 miles per hour, whereas engineers wanted to know the effects of 70-mile winds. He lived in hopes that a really stiff gale would blow one day in the right direction, north-west or south-east; but there had never been a 70-mile gale in that particular direction since the experiments were started. His own view was that as the speed of the wind became higher the uniformity of pressure became even more marked, and he believed that would hold good at 70 miles per hour.

Sir G. K. Scott-Moncrieff. Major-General Sir G. K. Scott-Moncrieff agreed with the President as to the value of the Paper, dealing with one of the great attacking forces of Nature which engineers had to consider, and about which their knowledge had continually to be revised. Thus, in connection with wind-pressures on roofs, what had been considered at one time to be axiomatic had been much modified by

Sir G. K. Scott-Moncrieff.

further observation and research. The Author's observations and deductions and his tabular statements would be very valuable to engineers who had to construct bridges both for roads and railways all over the world. He would not criticize the matter of the Paper, but wished rather to supplement the information therein, by reference to some experiences which he had had. On one point only in the Paper would he offer comment, and that was with regard to the shielding effect of the windward girder of a lattice girder bridge. It was amazing to learn that the pressure on the leeward girder was only 25 per cent. of that on the windward one. When the wind was not blowing quite at right angles to the bridge it would appear likely that the wind-pressure on the bridge as a whole would be practically double that on the windward girder. The figure given by the Author was very satisfactory, in that respect, and it was surprising that the shielding effect was so marked. Another thing that surprised him was that exceptional disasters due to whirlwinds and gusts were of comparatively infrequent occurrence. The number of accidents reported as due to the overturning of railway carriages on the plains of India was so small—no doubt they were all carefully recorded—that the Author seemed to be justified in considering them as negligible, or at least as a matter to be dealt with according to exceptional local conditions. In considering this subject it was necessary to take into account not only railways and roads in comparatively flat or undulating country, but in hilly country. On mountain railways bridges had often to be constructed at sites where the local conditions must be very carefully considered: wind-pressure was often of exceptional violence at such sites, and the height of the bridges was often very considerable. It was known that ground friction had a marked effect in reducing the violence of wind, and that the slope of the ground affected the local direction of the wind. Therefore, in the case of a high bridge spanning a ravine on a curve, there might be, and almost certainly would be, exceptional conditions of instability. For instance the Louise Margaret bridge over the Chuppar Rift, on the Sind-Pishin Railway was, if he remembered rightly, 290 feet from rail-level to the ravine below, and immediately above it there was a funnel-shaped gorge, down which the wind blew with extreme violence. The gauge of the line was 5 feet 6 inches, and although he did not think there had ever been any question as to the stability of trains passing over that bridge, yet if the gauge had been 2 feet 6 inches he thought that certainly exceptional measures would have had to be taken to ensure stability. Not only did the wind blow down such a gorge with great force, but the slopes of the ground on either side introduced an upward vertical

Sir G. K. Scott-Moncrieff. component of wind-pressure, which tended to reduce the effective weight of rolling stock passing over the bridge. Again, mountain bridges were frequently laid out on curves, the convex side of which was upstream and the super-elevation of the rails brought on the downstream side at low speeds of train stresses, due to the upward component of wind-pressure, which were of importance and should be taken into account. The same thing applied to road-bridges in mountainous districts, specially nowadays with motor traffic. One of the last bridges he had to construct before leaving India was across a gorge with vertical precipices on the sides, where the conditions were such that the bridge was exposed to wind of exceptional violence. The design was on the cantilever principle; and probably in that form and in suspension bridges, the danger was greater than in ordinary girder bridges. He had to consider not merely the loads coming on the bridge but also the upward vertical action of the wind, and the fact that motor-cars crossing the bridge in a violent storm would naturally take the leeward side, thus producing unequal transverse stresses. It was necessary, therefore, to guard against the possibility of cars running so near the leeward edge as to bring excessive stress on the cross girders, and consequently extra stresses on the main girders, in addition to the stress produced by the direct action of the wind on them.

Mr. Grierson. Mr. T. B. GRIERSON was not sure that there was much that was new in the Paper, but the Author had shown much industry in compiling it. The Author appeared to state that the Forth bridge was designed for a wind-pressure of 56 lbs. per square foot over the exposed surface, but that was not quite correct. Professor Rankine and Sir Benjamin Baker had stated that the Forth bridge was calculated to sustain a wind pressure of 56 lbs. per square foot striking the whole or any part of the bridge at any angle with the horizon and acting squarely or obliquely on an area equivalent to twice the plane surface of the front girders, with a reduction of 50 per cent. in the case of tubes. Even that was a very severe condition, but it was agreed to by the two Government inspectors and Sir Benjamin Baker himself, and there was a good deal to be said for the precautions taken in the interests of public safety by the inspectors, who were engineers of great experience and ability. He discussed the matter with one of them after the Tay bridge disaster and during the building of the Forth bridge, and during that period one bridge he had designed had to be altered slightly to comply with a new regulation of the Board of Trade. The bridge in question was over a river 600 feet wide and 30 feet deep at low water. There were four spans of 112 feet each, and an opening bridge in the middle, with two 40-foot spans and a turnstile bridge. He knew that there were frequently high winds at the site, and when walking across the bridge about 3 years later he was blown across the bridge against the lee lattice bars and was thankful that he had put in four systems of bracing as otherwise he would have gone into the river. He thought the requirements in connection with wind-pressure had been raised to an unjustifiable degree. Those who remembered the Tay bridge affair knew that engineers at that time believed the disaster to have been due to bad workmanship, and materials in the structure, through want of proper supervision by the company's engineering inspectors, and perhaps also to too small a factor of safety in some of the members of the bridge. Had the train not been derailed there might not have been any disaster. Before that he doubted whether many engineers made any special allowances for wind-pressure, except when calculating the stresses on the horizontal bracing under the cross-girders for one-fifth of the (test) rolling load. They allowed for snow and wind and other things in a general way. It seemed to him that something like a scare took place when Sir Benjamin Baker showed the high pressures he was recording. About that time he was asked to examine the roof of a circular building at Glasgow built for (and in use as) a panorama. The roof was of cupola shape, with radiating principals. He found that the roof was perfectly safe except that it needed wind-ties to make all the principals act together and to prevent local pressure from destroying them in detail. Afterwards he heard that the roof had stood safely through a gale when 56 lbs. per square foot was stated to have been registered at the Forth bridge. Adopting the Author's suggestions for Great Britain, the vertical component of the forces assumed to be acting on the structure in the case of one bridge he had designed, would be at least three times as much as the horizontal component due to direct pressure and therefore stability. The weight of the bridges alone when built to British Board of Trade requirements, made them so stable that no force of wind in the experience of this country could possibly affect them seriously. He had come to the conclusion that 56 lbs. per square foot on the total exposed surface as a basis for calculating wind-pressures in Great Britain was too high; that it had never been registered anywhere in this country; and that all the accepted formulas gave far too high results. Dr. Unwin's and Dr. Stanton's formulas differed very little from Smeaton's, which he thought showed that the older engineers also knew a good deal about the subject. Mr. Grierson.

Sir WILLIAM ELLIS remarked that, as an engineer, he had limited experience of wind-pressures, chiefly in connection with roofs. But Sir William Ellis.

Sir William Ellis. as an old mountaineer he had great difficulty in accepting some of the figures given in the Paper; and he had still greater difficulty in agreeing that the difference between mean pressure and gust pressure was so small as was stated and in accepting the statement that above 250 feet flying was safer, owing to less variation in the velocity of the wind. For the last 10 years he had always lunched some time at the beginning of the year at a place in this country which was 3,000 feet high, and others who had had the same experience would know what wind-pressure at such heights was. Even in the most stormy times the wind down below did not approach in violence that at 3,000 feet, and still less that at 13,000 or 14,000 feet. The danger there was not so much the force of the wind as the great difference between its normal force and its force during gusts. On the top of Helvellyn a steady gale could be withstood, but when gusts came anyone who did not instantly lie down was blown over; and he could not imagine that the difference between those forces was not much greater than 20 per cent. Once, when on a steep snow slope at about 13,500 feet, roped with about 20 feet of rope between himself and a guide, a sudden gust sheared off, as with a knife, from the apex of the snow slope, a piece of snow 12 inches wide and about 12 feet long, between himself and the guide. If Dr. Stanton would arrange to fit up his apparatus on the top of Snowden, he thought that the records obtainable would convince him that his figures needed considerable modification.

Mr. Wing. Mr. S. P. WING, as a member of the American Society of Civil Engineers, thanked The Institution for according him the privilege of taking part in the discussion. For the last 10 years he had been engaged in designing and building wireless masts up to 700 feet high. In the case of such masts the question of wind-pressure was obviously very important. He was much surprised at the Author's statement that 50 per cent. of the steel in the Forth bridge was needed merely to provide for wind-pressure. In wireless towers the steel needed purely to resist wind-stresses was as much as 80 per cent. of the whole; in other words, the wind-load was practically the only live load. The effect of assumptions made with regard to wind-pressure would therefore be easily imagined. There had been much discussion as to what loads should be assumed for such masts. A recent specification issued by the General Post Office called for a wind-pressure loading of 60 lbs. per square foot. That seemed excessive, but the thing that really mattered—and which the Author had brought out—was the velocity with which that pressure corresponded. Engineers had been content to accept the relation $P = 0{\cdot}003\ V^2$, and many used that assumption for buildings without

Mr. Wing.

great regard to what velocity really was involved. For example, a common wind-loading assumption was 30 lbs. per square foot, and probably many engineers would say, on the basis of that formula, that they made their structures safe for a wind velocity of 100 miles per hour. It was only when he saw the Paper that he realized that not only the shape but also the extent of the structure, and the effect of gusts, influenced the pressure; and perhaps that loading might only correspond with a wind-velocity of 80 miles per hour. That question had still to be decided, and it seemed to him that the general formula was defective. It was accepted as mathematically fixed, which it was not. It seemed to him a mistake to specify a wind-load of so many pounds per square foot. Suppose a bridge were put out to competitive design on the basis of providing for a wind-load at 30 lbs. per square foot: it appeared from the Paper that, if it was a through bridge and if the deck was covered, the corresponding velocity might be different from the corresponding velocity for another type of bridge, in which there was no decking and no plate girder; and thus the two bridges would have different factors of safety under the same wind. He thought specifications should say that the structure was to be designed for a wind of, say, 90 miles per hour—which seemed to him a reasonable assumption—and it should be left to the designer to justify his determination of the coefficient. That was easy now, for structures of a certain type, because models could be built and tested in a wind-tunnel, and in that way the coefficient could be determined, thus ensuring that competitive designs were calculated for the same wind-velocities.

Sir Francis Spring.

Sir **Francis Spring**, after mentioning his desire to associate himself with the last speaker in having been a member of the American Society of Civil Engineers for 27 years, remarked that the members were greatly indebted to the Author for his illuminating and interesting Paper. The Author had special qualifications for discussing such a subject, as he had been on the Indian Bridge Committee, which for some time had been investigating the wind and other elements of bridge stress. For the determination of wind pressure reliance was placed on certain formulas and instruments. He assumed that in the case of the cup anemometer the "observed velocity" which had to be multiplied by a coefficient in order to obtain the "wind-velocity" was the linear velocity of the cups on their circular path; but that was a point which should be made clearer. Cup anemometers, however, were not always quite trustworthy, being liable to the effects of the weather and of want of lubrication and attention. At Madras there were two cup anemometers, one at the Government Observatory, about 2 or

Sir Francis Spring.

3 miles inland, and another, his own, on the top of the harbour-office, right on the sea-front, about 50 feet above sea-level and with 600 miles of open sea to the east of it. Between the records of these two instruments there was a difference of 30 or 40 per cent., due, the Government Astronomer believed, to the velocity of the wind being reduced in its passage over 2 or 3 miles of heavily-timbered land before reaching the observatory. That was a matter to be taken into consideration. On the recommendation of Dr. Gilbert Walker, F.R.S., the head of the Meteorological Service in India, he obtained a Dines tube apparatus and put it on the top of a signal mast 130 feet above the sea and free from any obstruction. The aperture of the tube was kept always facing the wind by means of a weathercock. That Dines anemometer, which purported to have been duly calibrated, seemed to him to be as free from errors as it was possible to make a mechanical instrument, and probably its curve, which was drawn continuously, was as true a curve as could be obtained; and, as he had already said elsewhere,[1] it indicated considerably higher velocities than did either of the cup anemometers. He thought it probable, however, that the greater height of the records might be due to the fact that the Dines tube, because of its greater freedom from inertia, found it easier to record brief and sudden gusts than the cup anemometer. The Author quoted several instances of the blowing over of trains. Sir Francis Spring had described previously[2] the effects of a cyclone on telegraph poles and trains on the South Indian Railway. It was interesting to notice that, in those cases of trains being blown over, nobody, or hardly anybody, had been killed or hurt; from which it might be concluded that, with winds of that character, trains would nearly always be at a standstill. A train would not necessarily be blown to a standstill, but probably if it were not forced to a standstill, the driver would prefer to pull up. It was always possible, and would sometimes be advisable, to stop a train before it had begun to cross a long bridge in such a wind. He agreed with the Author's advice that the 56 lbs. per square foot allowed for in calculations might safely be reduced considerably. He thought it likely that 120 miles per hour was a velocity which might happen occasionally in India, where there were cyclones; but these were rare, and he doubted the necessity of making all exposed Indian structures heavy enough to withstand a wind which, at any given place, might occur only for a few minutes in a century.

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[1] Minutes of Proceedings Inst. C.E., vol. clx, p. 5.

[2] *Ibid.*, vol. ccx, p. 4.

Mr. Ellson.

Mr. **George Ellson** remarked that the Paper appeared to cover most of the possibilities which might arise in connection with bridges, particularly in exposed situations, from wind-pressure. The conditions in India seemed to be rather more difficult than those at home—apart from such exceptional bridges as the Forth bridge, where the conditions were abnormal owing to the height of the structure and its very exposed position. When the roof of Charing Cross station collapsed in 1905 it was calculated that, if the structure had at any time during the 40 years of its life been subjected to a horizontal wind-pressure of 27 lbs. per square foot, it would have collapsed, owing to the flaw existing in the main tie-rod which eventually caused failure. One had only to look at such things as walls, hoardings, and close fences to realize that wind-pressures generally could not be so great as had been assumed on theoretical grounds. Under the existing Board of Trade rules a horizontal wind-pressure of 56 lbs. per square foot had to be allowed for, and he believed the figure in the Ministry of Transport's proposed requirements was 40 lbs. per square foot. Under the Indian Bridge Rules, he believed, an excess of 25 per cent. of the total stress given by all other loads was permissible for wind-pressure; but such was not the case in this country. It was a remote possibility in the case of new bridges that the maximum load would proceed over a bridge at the critical speed and in a hurricane; but in the case of bridges 20 years old or more it was quite possible that, with the later types of locomotives, the maximum load which the bridge could carry safely would be reached frequently, and it was rather difficult to see what ground there was to justify the excess of 25 per cent. allowed by the Indian Bridge Rules. Judging by the results in the Author's Tables, it appeared that quite an average wind might result in a very considerable increase of stress. At the present moment he was rebuilding a bridge over the River Stour at Richborough in an exposed situation, and he had been informed that on two occasions during this winter the wind-pressure had been such that men engaged on sinking the cylinders had to cease work. It was a very ordinary type of bridge, of three spans with a middle opening of 55 feet, and consisted of a centre main girder, and two outside main girders and a trough floor resting on the bottom booms of the main girders. He calculated that a horizontal wind-pressure of 40 lbs. per square foot acting on the structure and on the highest vehicle that might pass over it, a vehicle built to the full size of the loading-gauge, would result in an increase of stress of 0·3 ton per square inch, or 4 per cent. of the total stress from all other sources. Such a low result was practically negligible, Mr. Ellson. and he was inclined to think that the bulk of the bridges of a similar type in this country would give about the same result. He mentioned that case because it compared with the 60-foot span bridge referred to by the Author in Appendix C. In that bridge the increase of stress due to the maximum wind-pressure was 2·54 tons per square inch, or 39 per cent. of the total. It seemed to him that a bridge of that type was of rather an unsuitable design for resisting wind-pressure. It might have certain advantages; but, when wind-pressure was taken into account, it might be found that the disadvantages outweighed the possible advantages. Judging from the Tables which were given, the deck beam type of bridge with a pair of main girders close together appeared to be rather common in India, where they were apparently used for spans up to 250 feet. That it was quite possible to design structures economically so that the wind-stresses should be low was also apparent from Table VII, where a through bridge of 150 feet span was mentioned, on which the maximum wind-pressure gave a stress of 22 to 30 per cent. of the total stress from all other loads. In another column of that Table a deck beam span of 200 feet was referred to, and there the maximum wind stress was only 6·5 per cent. of the total. Therefore it seemed that, if a little preliminary consideration was given to wind-pressure, a reduction in the stress therefrom could be effected by a suitable design which might in the long run be of benefit. He had read the Paper with great interest and pleasure, and congratulated the Author on making so valuable a communication to The Institution.

Mr. Moncrieff. Mr. J. MITCHELL MONCRIEFF, who remarked that he also had been a member of the American Society of Civil Engineers for many years, congratulated the Author on his Paper. It was not necessary to emphasize the importance of wind-pressure on structures. In most ordinary bridges, however, such as engineers were in the habit of dealing with, the quantity of steel required on account of wind-pressure was not large. The figures and facts given in the Paper, and the Author's method of looking at the subject, would be of use in another type of structure altogether. Those who, like himself, had dealt with giant cranes standing 200 feet above quay-level, with a huge "sail" on top of the tower, knew the anxiety entailed in regard to wind-stresses; but even in such structures he had found that if, after the work was done, he were asked how much bracing might have been dispensed with if there had been no wind-pressure, he would shrink in most cases from dispensing with any of it. When considering structures with regard to wind-pressure, the number of elevations taken into account should always be stated.

Mr. Moncrieff.

It was of no use to say that the wind-pressure was 30 or 60 lbs. per square foot unless there was some indication of the number of elevations on which the pressure was to be taken. He remembered discussing many years ago a Paper on the railway bridge over the River Tyne at Wylam, Northumberland.[1] It had a braced arch of 240 feet clear span. He had asked the engineer for the bridge, the late Mr. W. G. Laws, M. Inst. C.E., how many elevations he had taken into account when considering wind-pressure. His reply was "one." The vertical members from the arch down to the platform were flat bars 9 inches in width and 12 to 15 feet apart. He asked Mr. Laws (who was a man of unusually large dimensions) what shelter he got when he stood behind a lamp-post at a distance of 12 to 15 feet, and Mr. Laws confessed that he got no shelter at all. Mr. Moncrieff thought that two or more elevations should certainly be taken into account. The old Redheugh bridge across the River Tyne at Newcastle-on-Tyne,[2] which he had to condemn in 1895 after a very minute and careful investigation, was a very interesting structure. He believed that this bridge was the second most important bridge structure of the late Sir Thomas Bouch, the bridge of first importance being the ill-fated Tay bridge. The old Redheugh bridge ran almost north and south between Newcastle-on-Tyne and Gateshead; thus it faced the Tyne valley and was frequently exposed to violent storms. There was no wind bracing, properly so-called; in fact, between the main girders there was no wind bracing whatever. There were two flimsy cross-frames overhead in each span, of very light angle bars, between the top booms. There was no wind bracing at all between the bottom booms. The "wind girder" was provided by a wooden floor 5 inches thick in two thicknesses, 3 inches longitudinal and 2 inches transverse and 20 feet wide, and by the footpaths. The main structure of the bridge was about 840 feet long, with two central spans each of 252 feet and two end spans each of 168 feet with arched masonry approaches on each river bank. The height of the top of the main girder above low water was about 128 feet. These main girders were 22 feet 6 inches deep for the greater part of their length, but reduced gradually to about 14 feet at the shore ends of the 168 feet spans. The main girders were supported at two points in each span, these points being at about one-third of the span from the piers, and the support was given by large flat inclined ties. These ties ran from the bottom booms of the main girders to the tops of braced

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[1] Minutes of Proceedings Inst. C.E., vol. lvi, p. 262.

[2] *The Engineer*, vol. lxxxiv (1897), p. 23.

Mr. Moncrieff. towers which stood upon the heads of the river piers. The height of the tops of these towers was 170 feet above low water-level. The width of the braced piers of the bridge was only 23 feet at their heads to 26 feet 9 inches at the tops of the masonry columns on which they stood. These braced piers consisted of cast-iron pipes braced together with flat diagonal bars, with cast-iron struts between at vertical intervals of 13-foot to 14-foot centres apart. Those flat bars were 4 inches by $\frac{3}{4}$ inch from top to bottom, and each end had a cotter-hole cut in it 4 inches long by 1 inch wide. He calculated the wind area of the bridge, and, taking 56 lbs. per square foot on two elevations, the bridge could have never stood; and yet it had been there from 1871 to 1896 and had survived great storms. In examining the bridge he found that one of the diagonals in the bottom bay of one of the towers was useless, and that left only one diagonal in the bottom bay, where the stress was greatest, just above the level of the top of the protection jetty which surrounded the base of the river pier. Taking that point as a basis for investigation, he found that that bridge would have been blown over by a wind-pressure of something less than 19 lbs. per square foot on two elevations; he did not know how much less, but he knew it would be less because below the braced pier itself the bridge stood on four stone masonry legs of circular section about 22 feet high above the tops of the cast-iron cylinders which formed the foundations. Those stone legs were 7 feet in diameter at the top and 8 feet in diameter at the bottom and the two pairs of legs were only 26 feet 9 inches apart, centre to centre, and they had no bracing whatever between them. The top of the main girder was 128 feet above low water. That bridge formed one of the largest wind-gauges he knew of. It had been tested in the North of England for 25 years, and the result proved that there had never been a storm from east or west in which the wind-pressure equalled 19 lbs. per square foot on two elevations of the calculated exposed area. Whilst designing the new bridge[1] which replaced this structure he was much exercised in his mind with regard to wind-pressure, and did not feel at all easy about anemometer records. He could find no general agreement or confirmation as to how the anemometer velocity-record could be related to the wind-pressure. He looked upon a tall bridge like that which he had described simply as something standing on its base which might blow over, without bringing in any question of inertia, or screening, or anemometer-springs, or anything of that kind. He therefore desired to compare it with something which would also blow over, and he had four or five boxes prepared, about 18 inches square and 4 inches thick, with a hinge on one edge at the bottom, so that with definite wind-pressure per square foot the boxes might blow over. He had the boxes placed in such a way that they would blow over with clearly defined wind-pressures per square foot on their faces—5, 10, 15, and 20 lbs. He had the boxes calibrated to resist such pressures and the hinges were lubricated from time to time to make sure that they would act quite freely. The 5-lb. box was often blown over, the 10-lb. very rarely, and the 15-lb. not at all during 5 years. Those boxes stood on the top of a masonry pier on the south side of the river Tyne, nearly 110 feet above low water-level, and in a very exposed position. The Author had raised the question whether wind-pressure should be treated as live load. He himself had not treated it as such. If wind was treated as a live load it would require an increase in sections, assuming that the dynamic theory really accounted for the failure of materials used in bridge structures. If, on the other hand, the question was treated from the point of view of range of stress and resulting fatigue of material, he would certainly make no allowance whatever for wind-pressure, because, in carrying out experiments on range of stresses, it was well known that before a piece of material could be caused to fail it might be necessary to have a repetition of stress ranging from thousands to millions of times. There would not be thousands of great storms to consider in connection with any structure built by engineers, and therefore he took the chance of a very low factor of safety as far as that was concerned, and would treat wind load as dead load as a general rule. Mr. Moncrieff.

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[1] *Engineering*, vol. lxxi (1901), p. 608.

Mr. R. H. Read pointed out that an anemometer placed on the top of a structure in a exposed position would not necessarily be suitably placed for obtaining accurate records. During the building of the Forth bridge he had stood two or three times on the top of a main pier, when there was a very high wind blowing, without feeling any noticeable pressure of the wind, the reason being that the wind, striking the pier, was deflected over his head. A piece of paper tossed above his head would fly away at 40 miles an hour, but he could stand without feeling any pressure. An anemometer placed immediately on top of such a structure would give unreliable date, as a wind of low velocity probably would record a higher pressure than a wind of high velocity passing above the instrument. Mr. Read.

Mr. W. H. Thorpe questioned the validity of Mr. Box's statement as to the dynamic action of quickly-rolling loads on beams, which the Author quoted. The load falling freely, as assumed by Mr. Thorpe.

Mr. Thorpe. Mr. Box, had mass, but no weight, and exerted no pressure; so that the reasoning presented involved deflection of a beam by a force which had no existence. Mr. Box supported his statement by reference to certain experimental results, but, if examined closely, those results failed to confirm his statement. The experiments in questions were made in a beam 27 inches in length and ¼ inch thick, or about 108 times as long as it was thick. Experiments made upon such very flexible beams could not be considered applicable to ordinary girder work. A load running on to such a beam ran down the flexure curve until it reached the middle where, charging into the rising curve, the pressure on the beam was increased and also the deflection. In a beam of this character the deflection might thus be more than twice the static deflection. It was even conceivable, considering the inertia of the beam, that the maximum deflection might occur after the load had left the span. More relating to that subject could be found in Barlow's "Treatise on the Strength of Timber . . . and other Materials" (ed. 1851), where Professor Willis's investigation of experiments made 75 years since on the behaviour of loads travelling over flexible bars could be found. Mr. Box's conclusions were at fault, and the reasoning based upon those conclusions was thus vitiated. The Author said nothing about the possible effect of a rhythmic succession of gusts setting up oscillations in the structure. Mr. Thorpe did not, however, think that was a serious menace, because, to be effective, the frequency of the blows must match the time of vibration of the structure; but the effect of periodic impulsive forces on structures had been long suspected: there was, indeed, a legend with reference to the Coalbrookdale bridge (the first cast-iron bridge built in this country) to the effect that a strolling fiddler, chaffed by the men erecting the bridge, threatened to bring it down, and actually succeeded, by vigorous fiddling, in putting it into such a state of vibration that the men begged him to desist. The story was probably invented to illustrate a doubt entertained by some. On one occasion, when examining a timber footbridge, he noticed that the top boom of the main truss, about 40 feet in length, was unusually light. Pressing against it, he found that it flexed appreciably, and on repeating the pressure rhythmically the boom at last developed so considerable a movement that for fear of consequences he forbore. Apart from the possible cumulative effect of gusts on light structures, he suggested that, as no stress was developed in a structure without movement, and as movement from rest implied the resistance of inertia, it would appear that heavy structures, subject to maximum wind forces of limited duration, probably did not develop stresses comparable with those which might occur in light constructions. That indicated that it would be proper to take for very heavy structures a proportion only of the actual wind-pressure, the pressure inducing maximum stress being an inverse function of the mass. For a sufficiently heavy structure mass might be a complete protection against maximum wind forces, except as to the lighter parts which should satisfy full pressure requirements. He thought the effect of wind on structures was exceedingly obscure, and beyond the power of mathematical analysis; so that engineers would probably, as in many other matters, have to rely on observation and experience as the basis of rules partly rational and partly empirical. Mr. Thorpe.

Mr. G. E. W. CRUTTWELL remarked, with reference to Dr. Stanton's experience with wind-gauges at the Tower bridge, that before 1901 there was on top of the high-level foot-way of the bridge a Dines anemometer, and during 5 or 6 years the maximum pressure recorded by the instrument was 11 lbs. per square foot. During the same period the cup anemometers at Greenwich recorded 33 lbs. as a maximum. He had been very surprised to hear that Sir Francis Spring's experience was just the reverse. It seemed that such instruments were not at all reliable, and he thought it would be a good thing to be a little sceptical as to the formula $P = 0{\cdot}003\ V^2$. He was not convinced that the coefficient was correct, although he did not know any method of getting a more accurate one. The Forth bridge was not wide compared with its great span and there would be higher wind-stresses, and consequently a larger quantity of metal required than in a wider bridge. A bridge which he designed across Sydney harbour had a span of about 1,200 feet and carried two railway-lines, two tramways, and two footways, the width being about 100 feet. At the centre of the truss the leverage was very large, and yet the metal required to resist the wind-stresses on the whole bridge did not amount to more than 4 per cent. of the total. The Connel Ferry bridge on the Ballachulish extension of the Highland Railway had a span of 524 feet, and carried a single line with a footway about 6 feet wide. The trusses were only 21 feet 6 inches apart from centre to centre, and wind-pressure formed so large a proportion of the total stresses that special booms were provided, splaying out from the ends of the cantilevers towards the main posts, and increasing the width between centres from 21 feet 6 inches to 38 feet; and that was found to be a far more economical design than one with the trusses parallel throughout. Therefore a great deal depended on the width of a bridge. He agreed with the Author that the allowances of 56 lbs. on unloaded structures and 33 lbs. Mr. Cruttwell.

Mr. Cruttwell. on loaded structures were higher than were necessary in an ordinary railway bridge. He thought a fair allowance would be that adopted in Canada and Australia, namely, 30 lbs. per square foot. He did not see the reason for complicating matters by having one pressure for unloaded and another for loaded structures. An unloaded structure, if strong enough to take the stresses of its loads, would be strong enough to take the wind stress when there was no live load on it. Instead of taking the allowances given in Table V, he thought it would be quite sufficient to adopt a certain pressure merely for the loaded structure. The Author gave allowances ranging from 36 lbs. per square foot for a 40-foot span down to 31 lbs. for a 1,000-foot span. They were fairly accurately represented by the expression $\left(30 + \frac{40}{\sqrt{\text{span}}}\right)$ lbs. per square foot. That gave for a 40-foot span 36⅓ as compared with the Author's 36; for a 100-foot span 34 as against the Author's 34½; and for a 1,000-foot span 31¼ against the Author's 31. It would be applicable to any span.

Mr. Wilson. Mr. J. S. Wilson thought the Author underestimated the flange-resistance effect in deducing a tractive resistance, due to this cause, of 6 lbs. per ton of side pressure. That seemed to be far too small. Trains were pulled up more or less frequently by the effect of side wind. In a Paper[1] read before The Institution in 1908 by Mr. C. A. Carus-Wilson, the friction due to the flange grinding on the rail was determined by adopting a coefficient of 9·3, which would give a resistance of 680 lbs. per ton of side pressure. The Author used also 0·25 as a coefficient of friction between a beam wind of 90 miles per hour and the side of a train travelling at 30 miles per hour, which would give 560 lbs. tractive resistance per ton of side pressure. Assuming the velocity of the train to be 30 miles an hour, Mr. Carus-Wilson took 79 lbs. per thousand feet of area, which would give 622 lbs. total resistance for the train. Rather an interesting point came out during the building of the Forth bridge. When one of the landward cantilevers was nearly up to the abutment, it was thought that such an overhanging elastic arm 500 feet in length would show considerable lateral movement under wind-pressure. It was calculated that a pressure of 56 lbs. per square foot would produce a deflection of 11 inches. The actual deflection, due to wind-pressure, however, never exceeded ½ inch; which bore out Sir Benjamin Baker's views as to the effect of wind in gusts in different localities. Dr. Stanton's anemometer and Pitot-tube measurements on the Tower bridge seemed to indicate that high pressures extended over large areas simultaneously; but comparing **Mr. Wilson.** $\frac{1}{4}$ inch with 11 inches, the inference would be that there had never been, during the period of the observations at the Forth bridge, a higher pressure than $\frac{1}{40}$ of 56 lbs. per square foot on the whole area of the cantilever. It had been rather suggested in the discussion that Sir Benjamin Baker recommended 56 lbs. per square foot; but in all his statements he made it clear that he considered that far too high, and he often supported his views by pointing to existing structures and the much lower pressures which would have wrecked them. Large church windows, for instance, which a very small pressure would have blown in, had stood for centuries. It would be very useful if Dr. Stanton would be persuaded to undertake a few experiments on the resistance of trees to wind. The age of trees could be estimated fairly accurately; they seemed to be able to resist all winds for a certain time, and then a storm came which blew them over; which seemed to indicate that exceptionally high winds occurred at very long intervals.

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[1] "The Predetermination of Train Resistance." Minutes of Proceedings Inst. C.E., vol. clxxi, pp. 232–233.

The AUTHOR, in reply, remarked that the evidence brought forward **The Author.** by Dr. Stanton as to the value which might be assigned to the lateral variation was extremely interesting and valuable. Owing to the infrequency of severe storms in the right directions to affect his instrument on the Tower bridge, those experiments had taken many years; and even now the results recorded did not include any storm in which the mean wind-velocity approached the 75 miles per hour which the Author concluded was the maximum mean velocity for the British Isles. It was really those maximum conditions which interested engineers. In the three worst storms recorded by Dr. Stanton's instrument the mean wind-velocities were about 35 or 40 miles per hour. That, however, did not detract from the interest of the experiments. It was his belief that in storms of the maximum velocities the variation from the mean would, if anything, be less and not more than that recorded in storms of less intensity. In whatever way Dr. Stanton's experiments were interpreted, they must be taken to point to the conclusion that the difference in pressure on small and large spans in Great Britain was not so much as many engineers had imagined in the past. It was necessary to get at the truth and not to back up pet theories, and therefore it was of the utmost importance that the question should be very carefully examined. It appeared to him to be very difficult to average the readings of a large number of pressure-tubes for any instant of time· To begin with, the scale of each record must be small. Those records must be automatic and continuous, and the record of each tube must run on for some hours. Under those conditions the

The Author. thickness of the line traced would be important. From the horizontal scale of a print of a record kindly sent to him by Dr. Stanton it would appear that 1 minute corresponded with a length of 1¼ inch. The line traced was about $\frac{1}{50}$ inch thick, so that its width represented about ½ second. That half-second might make a difference in taking out the average at any particular instant. Again, the method by which the different records were co-ordinated was very important. The Indian Railways Bridge Committee in their experiments on impact had had to take very special precautions to co-ordinate results. The fineness of the line traced was very important, and various expedients were used to get as sharp a record as possible. An embryo thorn from a cactus leaf had been tried as the scribing-point, and the record had been taken on smoked glass or smoked paper. A cactus thorn was so fine as to be difficult to see. The thorn was given up in favour of a fine steel point made from a watch-spring; but difficulties were experienced in getting the line as thin as was desirable, owing to some of the smoked film piling up before the needle-point. A silvered surface gave good results. The diagrams were co-ordinated by electrical means, all the needles being lifted simultaneously by electro-magnets at intervals when an electric circuit was made by a brush on the cow-catcher of the locomotive used for the tests. Details of the instrumental difficulties and how they were met could be obtained from the Third and Fourth Reports of the Indian Railway Bridge Committee. It was evident, however, that to introduce the accuracy demanded in bridge impact tests into tests on wind-pressures would be a matter of great difficulty, In bridge tests the investigator could see when the train was coming. set his instruments going, and stop them when the train had passed. The horizontal scale might therefore be large, and the degree of accuracy accordingly greater. In wind tests one could not see the gusts coming and could not expect to get comparable accuracy. It was difficult also to take out averages correctly unless the diagrams given by the different pressure-tubes were co-ordinated by electrical means at frequent intervals. Was it certain that Dr. Stanton had avoided possible errors from those causes? If the diagrams were co-ordinated every minute or half-minute it seemed scarcely enough. All the diagrams might show the effects of a gust at, say, approximately the half-minute, but the peaks might vary by half a second or more; and owing to the closeness of the scale, the thickness of the line, and possibly looseness of some of the scribers, there would be a tendency to take all the particular peaks instead of taking the actual instantaneous values, many of which would be somewhat below the peak level reached. That might make some difference, The Author. but he did not think it would make a very large difference. Still it was a point to which he would like to draw Dr. Stanton's attention. He trusted Dr. Stanton would soon get a hurricane at the Tower bridge which would help him to arrive at final conclusions. With regard to the observations described by Dr. Stanton, presumably, in giving the average pressures, the number of tubes in each cluster was taken as one, and not six. The high pressures indicated on the cluster seemed only to exceed the high average pressures indicated by all the tubes by 6 to 7 per cent. In estimating the value to be assigned to the lateral variation of pressure, was it right to compare the record on a cluster at a selected point with the average reading of all the tubes? Would it not be more accurate to compare the average pressure of all the tubes with the maximum pressure recorded at that particular instant by any one of the tubes? If the cluster happened to be where the maximum instantaneous pressure was recorded, would not the lateral variation appear to be much greater? The Tower bridge had a span of 230 feet, and the lateral variation on that span appeared to be in the neighbourhood of 6 or 7 per cent. below that indicated by the cluster. Interpolating from the figures given in the Paper, the pressure on an unloaded span of 230 feet in Great Britain should be about 27 lbs. per square foot, corresponding with a lateral variation of about 30 per cent. In the case of a 1,000-foot span, his lateral variation worked out to 59 per cent. of the maximum gust pressure. Mr. Adam Hunter, M. Inst. C.E., had given him a copy of Arrol's "Bridge and Structural Engineers' Handbook" in which (p. 185) records of fifteen great storms between 1901 and 1906 at the Forth bridge were given. Five pressure-gauges, of the same type, were fixed on the Forth bridge, distributed over a length of 1 mile. The average readings of those gauges were 28, 23, 50, 13 and 30 lbs. per square foot. The mean of the readings was 29 lbs., or 58 per cent. of the average reading of the maximum gauge. The 58 per cent. did not really indicate the value to be assigned to the lateral variation; that value was obtained by comparing the maximum pressures recorded by the different gauges during the different storms. If the pressures could have been read and compared at particular instants, the mean reading might have been substantially less. An American had said that a wind gust had the slapping power of a wave. When Dr. Stanton had said that he thought the allowance for lateral variation could not be a large one, the remark had hit him with the slapping power of a gust. Dr. Stanton was an authority on wind-pressure. He imagined that if Dr. Stanton erected his instrument on Snowdon, as suggested by Sir William Ellis, he might get a larger lateral variation than he would

The Author. at the Tower Bridge, where possibly the river rather concentrated and averaged up the wind-stream filaments before they reached the bridge. He thought, however, in view of Dr. Stanton's remarks, that it might be wise to increase somewhat the pressures he had suggested for British practice. Turning now to Indian wind-pressures, with reference to which the Paper had really been written, the conditions there were somewhat different. Tornadoes and cyclones occurred and had to be allowed for. In tornadoes, although the area of maximum effect might be ½ or ¾ mile wide, there was stated to be generally a well-defined path of very high pressure, which was limited in breadth to a few hundred feet. Mr. Shaler Smith who had traced the paths of many tornadoes in the United States was credited with saying that in every case he had investigated, except one, a width of 60 feet would have covered the portion of the path in which the pressures exceeded 30 lbs. per square foot. Many investigators had concluded that it was improbable that tornadoes would ever exert a pressure of more than 30 lbs. per square foot over a length of more than 150 to 200 feet at a time. Therefore he thought it was justifiable to conclude that, even if the coefficient of lateral variation was generally shown to be considerably smaller than he had assumed for English conditions, it was probably of some importance in India and other countries, where tornadoes were likely to occur. Sir George Scott-Moncrieff appeared to think that there had been very few cases of trains being blown over by the wind in India. The Technical Paper (No. 147) to which he had referred was issued in February, 1904. Since that date many other wind accidents had occurred, but the full details had not been published. Probably in India a train was blown over once in every 2 years. Structures on mountain railways and in other exceptionally exposed situations would have to be considered on their merits. That Paper did not deal with bridges exposed to such very special conditions. In the case of a bridge in a funnel-shaped gorge 290 feet high above ground-level, the ordinary wind-pressure for the span-length considered might with advantage be increased, according to some formula such as that suggested by Mr. S. P. Wing, namely, $P = (0{\cdot}00126h + 1{\cdot}16)\,Pg$, where h denoted height above ground in feet, P the pressure in pounds per square foot at height h, and Pg the pressure in pounds per square foot at ground level. According to that formula, if Pg equalled 40 lbs. per square foot, the pressure at a height of 290 feet would be 61 lbs. Mr. Wing's formula had been developed for wireless masts, and would be applicable only to the case of a very high bent in the centre of a high and long viaduct. There were probably considerably higher wind-velocities on high

The Author.

mountains than on the plains or in low-lying country. The wind-velocities on mountains might nearly approach the "gradient velocity," or velocity of the upper air above the effect of ground friction. There appeared to be no definite rule connecting wind-velocities on the ground or at sea-level with the gradient velocity, which might be reached at 1,500 or 2,000 feet above the ground-level. Near the sea the gradient velocity might often be twice as great as the ground velocity. With low ground velocities the difference between the two might be considerably more than that. At the Eiffel tower, in light winds, when the velocity at ground-level was about 5 miles per hour, that at the top of the tower might be nearly 20 miles per hour, and the gradient velocity be four or five times the ground velocity. On the other hand there were indications that when really high wind-velocities were recorded on the ground, the gradient velocity might not be anything like double. Even when there was only light wind or no wind at all in certain sheltered valleys in mountain regions, if one climbed to the top of one of the highest mountains in the neighbourhood, one might have suddenly to face a wind-velocity not far short of the gradient velocity. The formulas given by Dr. Simpson, connecting the average maximum wind-velocity with the maximum and minimum gust-velocities, seemed to be reasonably accurate. On a small object like a man the maximum gust-velocity was approximately twice the minimum gust-velocity, but the pressure exerted by the wind varied as the square of the velocity. A man exposed an area of 6 or 7 square feet to the wind, and the pressure would vary, in a 50-miles-per-hour wind, between 15 and 60 lbs. per square foot. That pressure in an exposed and possibly dangerous position would appear to be even greater, and would cause a climber to crouch down so as to reduce his exposed area and cling to the rocks. The pressure in gusts, varying by possibly 400 per cent., was what a mountaineer had to consider. A point which he wished to make quite clear was that the Paper did not claim to cover other factors than wind-stresses, e.g., forces causing the lateral vibration of railway bridges. He would be sorry to see the wind-pressures now specified in some railway-bridge specifications reduced for short spans, unless at the same time something was specified in their place which would call for sound rigid bracing of the girders. In Indian practice, where 56 lbs. per square foot wind-pressure on unloaded spans had been used, and the bracings designed accordingly, he had found the bracings too light to hold spans of less than 80 feet properly and efficiently together. In some cases for small plate-girder spans he had had to put in bracings two-and-a-half times as heavy as those

The Author. needed to resist a wind-pressure of 56 lbs. Where there was no clause in a bridge specification calling for the provision of bracing to resist the lateral vibrations of the structure under the engine load, he considered it would be unadvisable to reduce the wind-pressures commonly used on small spans. But to provide bracing by specifying a high wind-pressure was uneconomical and unscientific. With regard to the remarks of Sir Francis Spring, he believed 0·0029 was correct for a very small area like 2 inches by 2 inches, and 0·0031 for an area 10 feet square. As to the Dines tube indicating a higher velocity than an anemometer fixed on the roof of a building, the latter might be very much affected by eddies around the roof, and therefore an instrument on the top of a flag-staff might show a much higher velocity than one on the roof itself. Mr. Ellson had objected to 25 per cent. extra stress being allowed in wind-pressures. The idea was that wind-pressures were very occasional loads, and that it was not to be expected that maximum wind-pressures, maximum impact, and maximum train-loads would often all occur together; and therefore 25 per cent. was quite allowable. He did not think there was any need to worry about rhythmic occurrence of wind-gusts, because in impact experiments in India it had been found difficult to get critical speeds, although in that case there were distinctly periodic forces, due to the engine. What Mr. Cruttwell had said with regard to a single pressure for both loaded and unloaded spans was sound.

Correspondence.

Mr. Cornick. Mr. H. F. Cornick remarked that the increase in wind-velocity which occurred at great heights above ground-level had been determined at the Forth Bridge on various occasions. Records would be found in Arrol's "Bridge and Structural Engineer's Handbook," where, also, experiments were cited which showed that the pressure varied approximately as the square root of the height above ground. It was obviously important to consider the height of the bridge itself above ground-level when deciding the maximum wind-pressure to be allowed for in the design. In addition, the configuration of the ground in the vicinity of the proposed bridge would need to be considered, having regard to the fact that many bridges, especially in India, spanned deep ravines or valleys, the "funnel" effect of which, in many cases, would produce a wind-pressure greater than that which would be obtained on level ground.

Mr. Cornick. He was unconvinced that no dynamic increment should be added for the effect of sudden rise of velocity in gusts. In the first place more data were required as to the "space area" of gusts and squalls, and as to the length of time during which gusts lasted. It was common knowledge that most destruction and havoc were occasioned by the wind when it came in violent gusts and squalls. The maximum stress in the girders of a bridge, as the Author pointed out, would only be reached if the increase of pressure were instantaneously applied and if it endured long enough to produce one complete vibration of the elastic body; but there were degrees of impact, and 100 per cent. need not be allowed. A gust of wind might be sufficient to carry a dynamometer fixed to a bridge to the full extent of its vibration, while, owing to its short duration, the same gust might have no appreciable effect upon the bridge as a whole. On the other hand, if the gust lasted long enough to cause the bridge to oscillate through the full extent of its vibration, it would cause the same stress as that indicated by the dynamometer. The oscillations of a tree in gusty weather showed that its maximum deflection was greater than that due to the actual wind-pressure at any moment, and it was conceivable that a series of gusts might synchronize with what might be called the oscillation-period of the bridge, so as to sway it, keeping time with its natural period of vibration, in which case very much greater stresses were likely to be involved, even though the wind had neither at any time reached the maximum velocity specified nor, on the other hand, been instantaneously applied. It seemed reasonable to make some allowance for sudden changes of wind-loads or pressures—just as allowances were made for the effects of stress-variations which were due to changes of load.

The experiments of Dr. Stanton and Sir Benjamin Baker related to the total positive and negative forces on two plates, i.e., the net resultant pressure on two plates, which were connected together and to a dynamometer, the wind being normal to the surfaces. If, however, the wind acted at an angle, it would have the effect of reducing considerably the shielding effect in the case of double bars in a lattice-girder bridge. According to Dr. Unwin's Table, the wind could vary 30 degrees from the normal to a surface without appreciably affecting the side pressure on that surface. Moreover, according to the Table quoted, wind at angles to the normal of 10–20 degrees produced greater pressures at right-angles to the surface than if the wind itself were acting normally to the surface; but with the wind acting at angles of more than 30 degrees to the normal its effective pressure at right angles to the surface was reduced. The question whether the reduced pressures on the semi-shielded bars

Mr. Cornick. would be more than in the case where the wind acted normally to the surface of the bars with others directly behind them would depend upon the distance apart of the bars and upon the angle at which the wind acted. In large bridges such relations would have to be considered. Dr. Unwin's figures in the Table were, he believed, strictly applicable only for determining wind-pressure on roof-surfaces and other box-shaped structures. The normal components of wind acting at various angles to thin plate surfaces would probably be higher, on account of the negative pressures involved on the rear of the surfaces, but the values would be likely to show the same ratio of decrease for increase of angle at which the wind acted.

Although impact due to live loads was an added vertical load, and the added stresses due to wind load were due to a horizontal load, they overlapped to a certain extent, reducing the stresses in some members and increasing those in others. Therefore one of the best reasons for specifying a reduced wind-pressure for a loaded span of moderate proportions was that full impact due to the live load was not likely to occur at the same time as the maximum wind-pressure.

He dealt with the case as follows: (1) He assumed the wind to be at a slight angle to the axis of the bridge so as to take effect on the exposed areas of the floor and of both windward and leeward girders and of any semi-shielded bars. (2) He assumed the wind to be in the same direction with a train on the bridge, and calculated the pressure on the train and on the exposed surfaces of the bridge only, that was, those parts unshielded by the train, but with a reduced wind-pressure. The pressure on the train was taken as a live load, to which a percentage depending on the length of the span was added for impact. The maximum stresses resulting from either condition were used in determining the necessary sectional area of the members required for resisting wind-pressure.

Mr. Larmuth. Mr. L. H. Larmuth remarked that the Author very rightly called attention to the heavy proportion of steel required to resist wind-pressure in past and present designs, as against that sustaining dead and live loads. He was to be congratulated on the information he had recorded and on the manner in which he had endeavoured to show that the practice as to the treatment of wind-pressures should be amended and placed on a rational basis. It might be assumed that all engineers were alive to the economy which would result by accepting lower wind-pressures in the case of engineering structures, but it was reasonably certain that a large proportion of them, probably the majority, were not prepared to make any radical change until absolutely convincing evidence had been produced to show that the pressures adopted at present were too high.

Mr. Larmuth.

He had assisted, in co-operation with a meteorological expert, in the selection of a number of sites for recording-stations for the Air Service, and it had been no easy matter to find sites which the experts would accept as being free from effects due to the contour of the ground, woods, buildings, etc. Those conditions governing the selection of a site for a recording-station were known to influence the records, and were a direct warning that local conditions should also be taken into account, as well as the records, when assessing wind-pressures. In his remarks on the maximum velocity in gusts, the Author briefly remarked on the effect of the increase in velocity due to height. The question of height was of the utmost importance in considering wind-velocities, for, according to the Monthly Weather Report of the Meteorological Office for 1918, the velocity varied approximately as shown in the following Table :—

Height : metres	½	1	2	3	4	5	10	15	20	25	30
Ratio to wind at 10 metres . .	0·50	0·59	0·73	0·80	0·85	0·89	1·00	1·07	1·13	1·17	1·20

At a height of 100 metres the ratio was understood to be 1·80. The instruments in the British Isles were placed at heights above the ground ranging from 16 feet to 105 feet. Some of them were placed on buildings, and it was recommended, in the report referred to above, that the heights of the buildings should be deducted when correcting results for altitude. It would be of interest if the Author could state the height of the instrument used at St. Louis in 1896, as that information, in conjunction with the figures given in the foregoing Table, might confirm the very high velocities quoted in the Paper in regard to the tornado at St. Louis in that year. The value of Table I in the Paper (p. 7) would be increased if information could be added, giving the approximate direction of the storm in each case relative to the railway, and whether the trains were on more or less level plains or on viaducts or embankments ; also whether the trains were moving, and, if so, with what velocity and direction. Instrument-records and the damage done by storms were invaluable guides, but it must not be overlooked that they did not necessarily indicate the maximum force of a gale ; it was impossible to place instruments where they were certain to be in the track of a gale, or where it might be assumed that the maximum intensity would be recorded. In the case of damage to structures, such as was recorded in the Paper, there was nothing to indicate how much the overturning-moment of the wind was greater than the resisting-

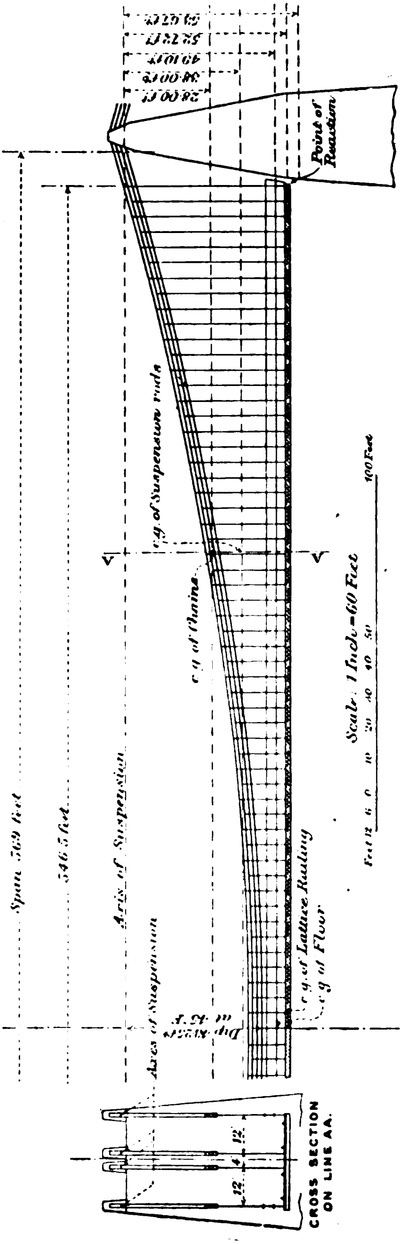
Mr. Larmuth. moment of the structure, and, if the real facts were known, it was not improbable that some of the cases given in Table I would have to be grouped with the two special cases which the Author considered abnormal. Those cases which appeared abnormal were a valuable warning to make haste slowly in the matter of cutting down the allowance for wind-pressure.

Mr. Morris. Mr. E. H. MORRIS observed that few structures existed which afforded better opportunities for gauging extreme wind-pressure over a large surface of open lattice-work than did the Menai suspension bridge. The bridge stood 100 feet above high water, and the bearing of its axis was north 30 degrees west, so that the whole suspended span was fully exposed to beam winds of the greatest intensity. The floor was freely suspended between the abutments, and the sole reaction to lateral wind-pressure was provided by a steel-plate buffer-girder bolted to the string-course, which engaged with the floor. In five heavy gales during the last 25 years the floor had broken adrift; two of the cases had been investigated, and the results served as a comment on the Author's experience. In a memorable gale on the 22nd February, 1903, both ends broke adrift simultaneously about 4.30 a.m. The failure occurred through the bending and twisting of the angle-bar riveted to the underside of the floor-plates and the shearing of some rivets. The resistance to crippling was so complex that, in order to arrive at an estimate of the force involved, he had constructed, a few years later, two models in steel, so proportioned that the strength in every direction was one-quarter of the corresponding strength of the parts that had failed. The models had been tested to destruction, and had failed by the shearing of the rivets at 7·5 and 6·25 tons respectively. From these figures it was concluded that the horizontal pressure at the time of failure had amounted to 25–30 tons at each end, or a total mean value of 55 tons. The net area of the vertical surface of the bridge on one side, chain, suspenders, railings, and floor, was calculated to be about 3,000 square feet. The bridge might be considered as suspended about four parallel axes passing through the points of suspension of the respective chains, being free to swing laterally, subject to the control of the buffer beam at a distance of 53·67 feet below the axis (*Figs. 16*). If the sum of the moments of each element of the actual wind-areas was divided by that distance, an equivalent wind-area was obtained referred to the point of reaction. The equivalent wind-area was calculated to be 3,480 square feet; in making the calculation the net areas of the chains and floor were taken once, and the area of one line of suspenders, which were only 1 inch square, was taken four times. The

Mr. Morris.

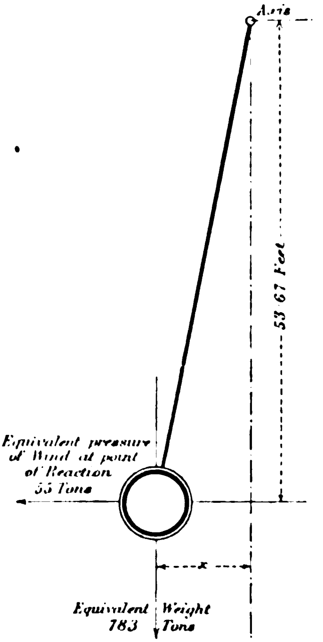
area of the leeward railing was reduced by 50 per cent. as allowance for interference by the windward parapet, which was of open lattice construction. (The net area of the parapet was 36 per cent. of the gross area.) The wind was assumed to be acting almost normally to the surface. Taking the total pressure of 55 tons as adduced above, the intensity of pressure on 3,480 square feet was 35·4 lbs. per square foot. When the bridge-floor was released from control it swung out laterally eastwards until the horizontal component of the weight of the suspended span, due to the obliquity of the chains and suspenders, became equal to the force of the wind. An approximate calculation of the lateral movement was made in the following manner. The weight of the structure was treated as concentrated in one plane at the point of reaction, and an equivalent weight was calculated by dividing by the axial distance (53·67 feet) the sum of the elements of weight, each multiplied by its respective distance in feet from the axis of suspension

Figs. 16.


Point of Reaction
c.g. of Suspension rods
c.g. of Chains
Axis of Suspension
Span 569 feet
546.5 feet
c.g. of Lattice Railing
c.g. of Floor
V
V
Scale: 1 Inch = 60 Feet
Feet 12 6 0 10 20 30 40 50
100 Feet
12
4
12
CROSS SECTION ON LINE AA.

Mr. Morris. (*Fig. 17*). The total weight was calculated to be 945 tons, and the equivalent or referred weight, 780 tons. Taking moments about the axis of suspension, the distance x at which the weight of 780 tons would come to rest under a horizontal force of 55 tons was 3 feet 9 inches. It was observed that one end of the floor had scored the face of the abutment for a length of over 4 feet, which agreed fairly well with the calculated figure. Unfortunately, no record existed of the wind-velocity in the vicinity of the bridge; but the records of the Dines autographic recorder at Holyhead, which lay 20 miles west-north-west of the bridge, had been inspected. The maximum velocity recorded at Holyhead between 4.0 a.m. and 5.0 a.m. was 87 miles per hour. The direction of the wind was south-west by south. If that record were assumed to be applicable to the site of the bridge, the value of the constant C in the formula, $P = CV^2$, became 0·00466, a figure 50 per cent. in excess of that adopted by Dr. Stanton for plane surfaces. The angle of incidence of the wind with the plane of the bridge being about 65 degrees, the value of the normal pressure would be little affected, as the Author pointed out. In the years 1908–10 some alterations had been made in the bridge; the railings had been renewed in a lighter form, and a more adequate buffing-arrangement had been provided at the end of the floor. Details of those alterations had been already published.[1] The reaction to wind-pressure was now provided by the resistance to shear of four turned iron bolts, two at each end, which were designed to shear at 12·5 tons each. As a result of the alterations the wind-area of the bridge had been considerably reduced; it was calculated to be 2,720 square feet, the equivalent or referred net area becoming 2,950 square feet. In a severe gale on the 5th December, 1914, the two bolts at one end of the bridge sheared, liberating the floor. The accident occurred

Fig. 17.


Axis
53·67 Feet
Equivalent pressure of Wind at point of Reaction 55 Tons
x
Equivalent Weight 783 Tons

---

[1] Minutes of Proceedings Inst. C.E., vol. cxc, p. 293.

at 6.45 p.m. The autographic records at Holyhead showed three gusts of 79 miles per hour in quick succession at 6.25 p.m. The direction of the wind was about south-west by west, exactly at right angles to the axis of the bridge. Assuming that that record was applicable to the site of the bridge, and applying the value of the constant C previously obtained, namely, 0·00466, CV^2 became equal to 28·6 lbs. per square foot, and the pressure on the two buffers of the bridge was 37·7 tons; in which case the two bolts failed at a pressure of rather less than 9·5 tons each. It might, however, be presumed that the severe strains which those bolts had undergone before the actual failure occurred had seriously weakened their resistance. It was not, of course, contended that the figures were in any way exact, but a marked measure of agreement occurred in the results, and that agreement suggested two conclusions:— Mr. Morris.

(1) That gusts of wind exerted at times a reasonably uniform high pressure over very large areas.

(2) That the reduction of pressure to 24 lbs. per square foot, as suggested by the Author, was unwarranted, at least for very large lattice spans, even in English practice, except in conjunction with a high factor of safety.

In the majority of calculations a sufficient factor of safety was allowed in the strength of the members, but in cases involving sliding contact or resistance to overturning, not less than 40 lbs. per square foot should be allowed for wind-pressure in exposed situations in Great Britain. The calculated wind-area of the bridge in each case was composed as follows:—

	1903.			1914.		
	Single Net Area.	Factor.	Referred Area.	Single Net Area.	Factor.	Referred Area.
	Sq. Feet.		Sq. Feet.	Sq. Feet.		Sq. Feet.
Chains	790	1·0	413	790	1·0	413
Suspenders	195	4·0	552	195	4·0	552
Railing	1,320	1·5	1,830	889	1·5	1,139
Floor	685	1·0	685	846	1·0	846
Total	2,990	..	3,480	2,720	..	2,950

Mr T. R. Nolan remarked that the Paper served a useful purpose in inviting the attention of engineers, experimenters, and theorists to the enormous importance of accurate information regarding wind- Mr. Nolan.

Mr. Nolan. velocities and pressures. That very little accurate information about this important subject was available was shown in the Paper. Even the experiments carried out by the engineers responsible for the designs of the Forth bridge appeared to have been faulty and to have exhibited exaggerated pressures. This question had presented itself very forcibly to him recently, as affecting the design of roofs for large carriage- and wagon-repair shops. The point had arisen what wind-pressure per square foot of vertical surface should be allowed for, and further, what reduction, if any, might be made for each succeeding roof-surface in saw-tooth roofs? Numerous well-known authorities, British and American, had been searched, but no definite and reliable information had been found. The methods given to guide engineers in designing roof-trusses and industrial buildings to resist wind-pressure were very detailed, but they made no allowance for screening effects or for the reduction in pressure per square foot on large areas which probably did take place. That effect was shown by the experiments at the Forth bridge. The result was that engineers continued to design roofs and buildings to resist stresses which they were never likely to experience. The most reliable figures for pressures due to the wind were probably those derived from calculations based on the overturning of trains in India. In Table I in the Paper the pressures calculated as required to overturn vehicles in India were interesting, and on the whole, probably very reliable; but it was noticeable that in only two of the thirteen accidents recorded did the pressures calculated exceed 31 lbs. per square foot. Those were the overturning of vehicles at Khana Junction in 1874 and at Ghazni in 1881, where the calculated pressures were 59 lbs. and 63 lbs. per square foot respectively. It was natural to question the correctness of the calculation of those pressures. The records of similar accidents on the Assam-Bengal Railway showed that "loaded goods-wagons" were overturned, but on further investigation it was discovered that the loading amounted only to something like 1 ton in a 10-ton wagon in individual cases. Was it not possible that in the Khana and Ghazni accidents a similar error had crept in? The Author stated that crowded passenger-trains on the metre-gauge might be held to be generally safe from overturning. On the 27th March, 1916, a mixed train, consisting of nine covered goods-wagons, five of which were empty and four lightly loaded, followed by seven bogie coaching-vehicles full of passengers, was overturned while standing at Chargola station, the engine having been detached for shunting purposes. The leading goods-wagon, which was empty, was only derailed, while the rest of the train, including two brake-vans at the rear, was overturned.

Mr. Nolan. It appeared that the overturning must have started with the loaded coaching-stock towards the rear of the train, otherwise the empty goods-wagon at the leading end would not have been left standing upright. He arrived on the scene shortly after the accident, and it appeared from evidence obtained that the train went over quietly and slowly. In fact, no one was hurt. That furnished fairly reliable proof that there were no gusts, but that the train was capsized under a steady pressure. In that case calculations showed that a pressure of about 30 lbs. per square foot would be sufficient to overturn the coaching-stock. It therefore appeared, so far as the metre-gauge was concerned, that it was unlikely that a bridge could be subjected to additional wind-stresses of more than about 30 lbs. per square foot due to passing passenger-trains, which were the only trains likely to pass over bridges in exposed positions at critical speeds, as such bridges were, as a rule, approached on heavy up gradients.

The Author The AUTHOR, in reply, observed that it was certainly very desirable to consider the local conditions, the exposure of the situation, and the height of the bridge. It was difficult to legislate for abnormal exposures, and the only course seemed to be to specify certain minimum pressures and to leave it to the judgment of the engineer to call for higher pressures in very high or very exposed bridges.

The lateral vibrations of a bridge structure were very rapid, and he did not think it was ever a question whether a gust would last long enough to cause the bridge to oscillate through the full extent of its lateral vibration, but rather whether, in the time-interval it took the wind to rise from its minimum to its maximum gust-velocity, the structure would naturally have completed 5, 10, 20 or more lateral vibrations. Obviously, if a gust took ½ second to increase from a velocity of 50 to 100 miles per hour, and the bridge in question would naturally complete 20 vibrations per second under the average side loading produced by the wind, the effect of the rate of rise of pressure due to the wind might be neglected, as it would practically be applied as a static load. He estimated that the 60-foot deck plate-girder span for the 2-foot 6-inch gauge under a side wind-pressure rising in gusts from 8 lbs. to 33 lbs. per square foot would tend to make 30 to 15 complete lateral oscillations per second; a similar broad-gauge span would have a much shorter period. In the case of a tree swayed by the wind the flexibility and deflection was very much greater, and the period of vibration much longer. A wind-gust could act on a tree in a way in which it could not act on a bridge.

Mr. Nolan would realize that it was often difficult to obtain accurate weights of vehicles blown over, because, for one thing, they

The Author. were unloaded before being lifted. Station records, however, might show their weights, when despatched. If, however, Table I was examined, it would be seen that at Khana junction the pressure needed to overturn vehicles was 59 lbs. per square foot. Those were broad-gauge vehicles; and Table II shows that loaded 5-foot 6-inch goods-stock needed pressures of 110 to 125 lbs. per square foot to overturn them: so that the inference was that even the heaviest vehicle was only partly loaded. At Ghazni Bridge (metre gauge) the pressures needed to overturn the vehicles ranged from 27·6 to 63·5 lbs. per square foot. Loaded metre-gauge goods-vehicles needed 65 to 90 lbs. per square foot to overturn them. There again the probability was that the majority of the vehicles were lightly loaded.

The accident at Chargola Station, mentioned by Mr. Nolan, was interesting; but because loaded passenger-coaches, needing 30 lbs. per square foot pressure to overturn them, might not be likely to approach a span at the critical speed, he did not think the possibility of goods-vehicles crossing at their critical speed should be overlooked. The maximum stresses in bridgework occurred, in all except small spans, under heavily-loaded goods-trains, and the critical speed for a goods-train was very considerably less than that of a passenger-train, owing to the smaller diameter of the driving-wheels of the goods-engine. For instance, a speed of only 25 miles per hour might be the critical speed for a metre-gauge goods-train on a 150-foot span bridge, whereas, under a passenger-train the critical speed would be possibly 35 miles per hour.

Apparently he had very much underestimated the flange-resistance due to the pressure of a side wind, as pointed out by Mr. Wilson. The likelihood of a train being pulled up in very high winds was much greater than the figures in the Paper indicated. At the same time, he did not consider that the assumption that trains would always be pulled up by the wind was justified, because in many cases when approaching an exposed bridge, they would be sheltered largely from the full effects of the wind by cuttings, hilly ground, towns, or even heavily-wooded country.

In the calculations of the 150-foot through span he had assumed that the overturning forces due to the wind-pressure on the upper booms was transmitted to the structure bay by bay. It was usual, where overhead bracing existed, to assume that the bracings transmitted the wind-stresses to the portals, and that they then passed through the end diagonals to the bearings; and that method, in the majority of cases, gave the most correct results. Whether any overturning-moment from the wind on the top booms was or was not transmitted would depend on the design and on the relative deformations of the

The Author.

parts. In a pony truss with no bracing between the top booms the overturning-moment due to the wind on the top booms must affect the whole structure. In the particular case in question the girders were very stiff laterally and had vertical members formed with solid webs and four bulb angles which would have transmitted some of the overturning-moments from the top booms bay by bay. It was assumed, therefore, that the span would act as a whole in resisting overturning-moments. If it were assumed that the upper wind-bracing transferred the whole of the wind loads on the top booms to the hips, and that it was then taken care of by the portals and end diagonals, it would reduce the overturning-moment stresses in the girders by 38 per cent., and there would be a different distribution of stress in the end diagonals. It would also slightly affect the horizontal bending stresses in the top boom. The leeward bottom boom would then come under a total longitudinal stress of about 3½ tons, owing to the outward thrust of the end diagonals, while the windward bottom boom would have a similar longitudinal compression developed in it. In the design in question the critical section was the leeward bottom boom, the critical member of which was the second bay from the end. If it were considered that no overturning-moment was transferred from the top boom to the span as a whole, then the wind-stress in this critical member would be reduced from 2·47 to 2·38 tons per square inch. The wind stress, therefore, would still be about 30 per cent. of the direct stress as stated. In the actual structure considered the stresses would lie somewhere between those computed by the two methods. In the 60-foot deck plate-girder span some designers would assume that the wind stresses on the upper boom were transmitted by the upper wind-bracings to the end sway-bracings, and then transmitted vertically to the bearings, and that only the overturning-moment of the wind on the train itself need be considered. There again the question was one of the relative deformations of the parts of the structure: there was little doubt that the intermediate sway-bracings would act so as to transmit the overturning-moments to the whole structure, and that the efficient system of bracings provided between the bottom flanges would help to deal with those stresses effectively.

Mr. Morris presented valuable information as to the action of the wind on the Menai suspension bridge. To interpret those observations the question of the effective areas exposed to the wind must be carefully considered. The double-track bridge was carried by four sets of chains. Each chain appeared to consist of four separate parts. The windward chains could give little, if any, The Author. protection or shelter to the inner chains, or the latter to the leeward chain. The inner pair were, apparently, three or four widths apart and the two should have a combined surface $1\frac{1}{2}$ times that of a single chain. The total exposed area of the chains would be, therefore, $3\frac{1}{2}$ times that of one chain. The suspenders being small, square rods, the area taken should be four times that of one set. The lattice railing was about 3 feet 6 inches high, and the two lattices were 28 feet apart. Even with a deck between, the windward lattice could give little shelter to the leeward one, and the full area of the two should be given. The resistance offered by the deck was more difficult to decide: it might vary considerably—between 1·0 and 2·0 times the side area exposed. It should not be taken as less than 1·5. Thus the following Table gave the exposed area in square feet in 1903:—

	Single Side Elevation.	Factor.	Total Area.	Referred Area.
Chains	790	3·75	2,965	1,550
Suspenders	195	4·0	790	559
Railing	1,320	2·0	2,640	2,415
Floor	685	1·5	1,030	1,012
Total	2,990	2·47	7,425	5,536

If it was assumed that in the storm on the 22nd February, 1903, the floor broke loose with a side pressure of 55 tons, then

the referred area × pressure in lbs. per square foot $= 55 \times 2{,}240$

and the pressure per square foot $= \dfrac{55 \times 2{,}240}{5{,}536} = 22\cdot25$ lbs.

The gust-velocity at the time was 87 miles per hour.

The mean velocity $(87 - 1\cdot5)\dfrac{1\cdot0}{1\cdot3} = 65\cdot8$ miles per hour.

The pressure at the mean velocity,

$$0\cdot0031 \times (65\cdot8)^2 \times 1\cdot35 = 18\cdot1 \text{ lbs.}$$

The pressure at the gust velocity,

$$0\cdot0031 \times (87)^2 \times 1\cdot35 = 31\cdot6 \text{ lbs.}$$

And the factor for lateral variation, $\dfrac{22\cdot25}{31\cdot6} = 0\cdot705$ of the maximum gust-pressure.

In the Paper he had taken the lateral variation for a 550-foot span, as $\dfrac{24\cdot53}{41\cdot84} = 0\cdot585$. The variation between the pressure at the mean velocity and the maximum gust-velocity was $31\cdot6 - 18\cdot1 = 13\cdot5$. In the case of the Menai bridge it was $22\cdot25 - 18\cdot1 = 4\cdot15$.

The actual pressure exceeded the pressure due to the average wind-velocity by 23 per cent. The Author.

The fact that the deck in swinging scored the masonry for a length of over 4 feet did not seem extraordinary. At the moment the floor broke loose a wind-pressure equivalent to a force of nearly 55 tons was acting on the bridge and kept on acting as the floor swung sideways. Under such conditions the maximum amplitude of the swing should have been nearly 3 feet 9 inches × 2 = 7 feet 6 inches.

In the gale of the 5th December, 1914, the bridge having been altered, the new areas were :—

	Single Side Elevation.	Factor.	Total Area.	Distance below Bearing.	Referred Area.
				Feet.	
Chains	790	3·75	2,965	28·0	$\frac{2,965\times 28}{53\cdot 67} = 1,550$
Suspenders. . .	195	4·0	780	38·0	$\frac{780\times 38}{53\cdot 67} = 552$
Railing	889	2·0	1,778	49·10	1,630
Floor	846	1·5	1,269	52·72	1,250
Total . . .	2,720	2·5	6,792	—	4,982

The force needed to shear through the bolts might have been 37·7 tons. The gust-velocity at Holyhead was 79 miles per hour, which corresponded with a mean maximum velocity of

$$(79-1\cdot 5)\,\frac{1\cdot 0}{1\cdot 40} = 59\cdot 5 \text{ miles per hour.}$$

The pressure due to the maximum mean velocity was

$$0\cdot 0031 \times (59\cdot 5)^2 \times 1\cdot 35 = 14\cdot 8 \text{ lbs. per square foot.}$$

The pressure due to the gust-velocity was

$$0\cdot 0031 \times (79)^2 \times 1\cdot 35 = 26\cdot 1 \text{ lbs. per square foot.}$$

The referred area × pressure per square foot = 37·7 × 2,240 lbs.

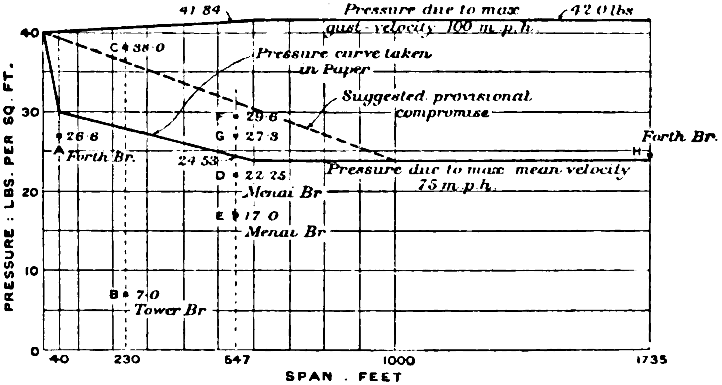
$$\text{The pressure per square foot} = \frac{37\cdot 7 \times 2{,}240}{4{,}982} = 17\cdot 0 \text{ lbs.}$$

The factor for lateral variation, $\frac{17\cdot 0}{26\cdot 1} = 0\cdot 65$ of the maximum gust-pressure. The actual pressure exceeded that due to the maximum mean velocity by 15 per cent.

Fig. 18 gave graphically the observed data. The full line indicated the pressures recommended in the Paper. Starting with a pressure of 40–42 lbs. per square foot, as equivalent to the maximum gust-velocity of 100 miles per hour (after multiplying by certain

The Author. coefficients to provide for the shape) on small areas, the curve dropped to 30 lbs. per square foot for 40-foot spans. The Forth bridge experiments indicated that on a span of that size the mean pressure was only 67 per cent. of that due to the gust-velocity. This value, corresponding with about 26·6 lbs. per square foot, was indicated on the diagram by point A. Although it was known now that the records of the gauges used at the Forth bridge were much too high, there was nothing to show that the relative readings of the different instruments were in error; and he felt, therefore, in the absence of any information to the contrary, that he would be justified in taking the pressure on a 40-foot span as 75 per cent. of the pressure due to the maximum gust-velocity. The line was carried straight from this value of 30 lbs. per square foot for the 40-foot span to 24 lbs. per square foot on the 600-foot span—corresponding with the pressure due to the maximum mean velocity of 75 miles per hour.

Fig. 18.



Dr. Stanton's recent experiments on the Tower bridge had led him to think that the factor for lateral variation on a 230-foot span could not be large. Apparently he believed it was only 6 or 7 per cent. below the pressure due to the maximum gust-velocity, and in this case it might be in the region of 37 to 38 lbs. per square foot. Dr. Stanton's Tower-bridge experiments were carried out in winds which gave a maximum pressure of about 7 lbs. per square foot, as indicated by point B. The corresponding point which would have been reached in a storm with gusts of 100 miles per hour was indicated by C.

In the case of the Menai bridge, according to the Author's interpretation of the results observed, pressures of 22·25 and 17·0 lbs. per square foot were reached on the 22nd February, 1903, and on the 5th December, 1914, respectively. If the wind-velocities at the

The Author.

bridge were similar to those recorded about the same time at Holyhead the calculated pressures exceeded the pressure corresponding with the maximum mean velocities (deduced from the maximum gust-velocity, according to the formula given) by 23 and 15 per cent. respectively. The pressures which he believed had been reached were indicated by points D and E on the diagram; and corresponding points, assuming storms of maximum intensities (namely, 100 miles per hour gust-velocity, or 75 miles per hour maximum mean velocity) were shown above the line, at F and G.

Finally, for the Forth bridge central span point H showed the lateral-variation factor of 59 per cent.—corresponding with 24·8 lbs. per square foot—to which he had referred in the Discussion. This, however, as already pointed out, was not the true value of the factor, which must be considerably less if the Forth bridge instruments gave reliable relative results.

In the interpretation of the Menai bridge observations it appeared that Mr. Morris had overestimated the shielding action of the windward parts of the structure. At the bottom of p. 13 the Author suggested that for ordinary open-web or triangulated spans with deck plates it was not safe to take the area exposed to the wind as less than 1½ times the total side elevation of the bridge. That suggestion referred to single-track bridges with two main girders. On p. 14 he pointed out that it would be advisable to allow for slightly greater areas for similar double-track bridges. For structures having three main girders it did not seem reasonable to allow less than twice the area seen in side elevation. These remarks referred to trusses having apertures of, say, 60 to 75 per cent. of their overall dimensions. In a suspension bridge the shielding action of the one part on the other would be less pronounced, and he did not think that for such a bridge, built for two tracks and having four sets of chains and suspenders, he had been over-cautious in suggesting that 2½ times the side elevation should be taken when calculating the area exposed. Mr. Morris had taken approximately 1·4 times the side area—and therein lay the difference. The fixing of specified wind-pressures must at all times be subject to reasonable interpretations of the area exposed to the wind. He did not think any modern bridge specification would allow only 1·4 times the side area, seen in elevation, of a suspension bridge having four sets of chains with flat links, and carrying a double track. In view, however, of the valuable data given by Mr. Morris, and of the opinion expressed by Dr. Stanton, it would seem advisable to take a conservative course and to adopt a line, such as that shown dotted on *Fig. 18*, from 40 lbs. per square foot for small spans straight to 24 lbs. per square foot for 1,000-foot spans. This represented the recommended

The Author. pressures on unloaded spans under English conditions, subject to a reasonably liberal interpretation of the areas considered to be exposed to the wind. For loaded spans the pressure would range from 35 lbs. to 22 lbs. per square foot. It would appear that under Indian conditions the pressures should range from about 56 lbs. per square foot on small unloaded spans to 33 lbs. per square foot on 1,000-foot spans and over, and on loaded spans from 45 to 31 lbs. per square foot.

Reviewing the situation as a whole, it appeared that in England the wind-pressures usually called for on unloaded spans might be considerably reduced, and that 40 lbs. per square foot need never be exceeded under ordinary conditions of exposure; while on large spans very large reductions—to less than half of the pressures now called for—might be specified. For small loaded spans the pressures now called for appeared to be reasonable; but in large spans they might be reduced to two-thirds of that specified for small spans. For Indian conditions it did not appear desirable to reduce the wind-pressure of 56 lbs. per square foot on small unloaded spans; but very great reductions might be made for large unloaded spans. In the case of loaded spans it appeared doubtful whether the 33·3 lbs. per square foot now specified was sufficient for small spans, and it would seem that the value should be raised by about 33 per cent. For really large loaded spans a wind-pressure of about 30 lbs. per square foot would appear to be satisfactory.

He agreed with the views that the design of the structure affected the question, that a due consideration of wind-pressures certainly did penalize designs in which the main girders were spaced close together, that the extra metal needed to deal with the wind in small structures was not large, and that even if wind was neglected altogether, any reduction in the weight of bracing was unadvisable. It was insufficient to proportion the bracings on railway-bridges of 250 feet span or less for resisting wind stresses alone. Such bracings should be proportioned to resist a vibratory stress, and, if wind loads specified for bridges in Great Britain were reduced, some rule should be introduced calling for heavier bracings, particularly in the smaller spans.

13 February, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The discussion on Mr. D. H. Remfry's Paper on "Wind Pressures, and Stresses caused by the Wind on Bridges" was continued and concluded.

20 February, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

(*Paper No. 4401.*)

"Colombo Drainage-Works."

By Richard Eustace Tickell, O.B.E., M. Inst. C.E.

Colombo is situated in latitude 6 degrees 56 minutes north and longitude 79 degrees 49 minutes east. It is bounded on the north by the Kelani river, on the west by the harbour and the Indian Ocean, on the south by the Dehiwela canal, and on the east by streams and swamps at sea-level, liable to floods up to 12–13 feet above sea-level. In the north and centre the ground rises to summits 80–100 feet above sea-level, but these elevations are separated by the San Sebastian canal, which runs from the Kelani to the harbour and passes, at about 6 feet above sea-level, through a lake of about 300 acres in the middle of the city. The extent of the city is about 8 miles north and south by 3 miles east and west. The total area is 8,617 acres, of which 5,725 acres are habitable; and the remainder consists of water, swamps, and other areas, such as parks and cemeteries, which are reserved against building.

The population is about 300,000, consisting of Sinhalese, Tamils, Moors, Eurasians, Europeans, and others; its density ranges from about 16 per acre in Kollupitiya ward, to 183 per acre in St. Paul's ward, and averages 52 persons per acre. The population has doubled during the last 35 years, but the growth is almost entirely artificial, over 90 per cent. of the increase being due to immigration. The south-west monsoon prevails from May to October or November, and the north-east during the remainder of the year. The climate is extremely humid. The shade temperature ranges from 62° to 97° F. and the monthly temperatures average from 79° to 83°. The average daily range is only about 12°. The temperature in the sun rises to over 160°. The mean rainfall is about 81 inches, and falls of 11·90 inches in 24 hours and 5·50 inches in an hour have been recorded. The city is administered by a municipal council partly elected and partly nominated by the Governor.

Before modern drainage was introduced, the liquid sewage stagnated in surface drains, until it was washed by the rains into the harbour, lake, canals, and swamps, or soaked away into the soil. With the growth of population, and following the introduction of an abundant water-supply in 1887, the pollution increased until the nuisance became intolerable. The contents of pails were supposed to be removed at night into so-called night-soil carts, and buried in a pitting-ground at Narahenpitiya in the south-east of the city, which afforded a prolific breeding-ground for flies.

The Drainage Scheme.—The drainage scheme was initiated by the Government of Ceylon in 1896, when the late Mr. James Mansergh, Past-President, Inst. C.E., was called in to advise. He tabulated the vital, topographical, and other statistics, produced the first contour map of the city, and recommended a complete sewerage scheme upon the separate system, with outfall works at the mouth of the Kelani river. His report, dated 10th December, 1897, gives a synopsis of all relevant local information available at that date; and although many circumstances have since occurred to modify the original plans, his report remains a monument of careful investigation, upon which the present scheme has been constructed.

The municipal council at first raised objections to any system of underground sewers, but in 1901, with some encouragement, not to say pressure, from the Government, the council consented to the commencement of the first instalment, which dealt with about one quarter of the city in the area draining towards the harbour; and subsequent extensions have followed in due course. Owing to a project for the construction of a large wet dock in connection with the harbour, which would cut through the line of the main sewer, further investigations on the ground were needed. In November, 1902, the Author landed in Colombo to report upon these questions, and to take charge of the works as resident engineer as soon as they could be organized and commenced. The sewerage scheme was then re-cast, and a new position was selected for treatment-works at Madampitiya, with an outfall into the Kelani about 2 miles above the mouth. In the meantime offices and stores were built, the nucleus of a staff was engaged, and plant and materials were ordered from England; but it was not until May, 1906, that the necessary land-purchase was accomplished for work to be started on the main sewer and treatment-works. The works were undertaken without a contractor, under the administration of the Government, and were financed by a Government loan advanced to the municipality. Fig. 1, Plate 1, is a drainage map of the area.

The Harbour Area.—Constructional works upon minor sewers in the Fort and Pettah wards were commenced on the 1st June, 1904. The ground was found to alternate between a hard gneiss rock and running sand. Over 2,000 tons of rock were blasted out of some of the busiest streets of the city, under tramways and beneath a network of gas- and water-pipes, old drains, and electric cables. The low-level harbour sewer (3 feet by 2 feet) was excavated in running sand and boulders along the edge of the old sea-beach, to a depth of 18 feet below sea-level, and included a tunnel under St. Anthony's Church, which stands on a promontory, into the old foreshore of the harbour. The harbour pumping-station was excavated to a depth of 45 feet below ground-surface and 20 feet below sea-level. The high-level sewer from the harbour pumping-station is 3 feet by 2 feet for a length of about ¾ mile to Layards Broadway, where it joins the principal main sewer from the southern part of the city. The main sewer is 12·66 feet below mean sea-level at this point, and 6 feet by 4 feet in section. It is a little over 1½ mile in length, and passes through a ridge of high ground at a depth of 43 feet below the surface; it then runs into the Madampitiya swamp, where the excavation was carried out in liquid peat mud, with an invert-level at the northern pumping-station of −20·25. An inflow of 150,000 gallons per hour of subsoil water had to be dealt with in this excavation.

Northern Pumping-Station and Treatment-Works.—These works were designed to deal with the sewage from the whole city, but subsequent enlargements of the city-boundaries necessitated the provision of a southern outfall, as described later. The sewers take six times the dry-weather flow, reckoned at 25 gallons per head of the ultimate population. The treatment-works and engine-power were to be constructed in four instalments, as occasion demanded. The first instalment dealt with a dry-weather flow of 1½ million gallons a day. A wet-weather flow, up to twice the dry-weather flow, was treated in septic tanks and sprinkling filters, and a volume of four times the dry-weather flow was dealt with in storm tanks. Much larger quantities however were actually put through them before a second instalment was constructed.

At the pumping-station the sewage flows through a silt-pit 9 feet 3 inches below the sewer-invert, worked by a revolving bucket and winch. It then passes into a screening-chamber, ventilated by a shaft 6 feet square internally, carried up 27 feet above the ground in the form of a tower. The screening-chamber is divided down the centre, and tandem screens are provided on each side; the coarse screens have 1¼-inch spaces and the fine screens ¼-inch spaces. They are cleaned by mechanical rakes on endless chains.

The pumping-station building is 144 feet by 67 feet at ground-floor level. The level of the sump invert under the dry-weather pumps is −25·00, and under the storm pumps −21·00. The pump-floor is at −10·50. The rising mains are carried in side galleries at + 5·50, and the engine floor is at a level of + 15·50. The building will accommodate eventually nine dry-weather pumps and nine storm pumps, with the corresponding engines. The first instalment consisted of three centrifugal pumps, with 8-inch suction and delivery, and three with 12-inch suction and 10-inch delivery, together with six 28-HP. suction gas engines.

Four septic tanks were constructed, with water-level at + 21·50. Each tank is 240 feet by 14 feet by 9 feet deep, a capacity equivalent to 12 hours dry-weather flow. These tanks are preceded by grit-chambers 27 feet long. They were at first roofed over with corrugated sheeting; but, as they bred mosquitoes, the roofing was removed, and the mosquitoes disappeared. The sludge from the tanks is discharged to low ground on the east side of the works. The effluent from the tanks is distributed to eight continuous-flow filters, each 100 feet diameter by 7 feet deep, with revolving arm sprinklers. The total effective area is 6,800 square yards, giving a rate of 220 gallons per square yard on the daily dry-weather flow. Two storm-water beds were constructed at + 12·50, each 165 feet by 85 feet by 7 feet deep, with half the capacity filled with broken stone. The effluent from the works is discharged into the Kelani river, through two cast-iron pipes 48 inches in diameter, the invert being at −4·25.

Rain-Water Drains.—A complete system of separate rain-water drains has been laid down, designed to deal with discharges of 2 inches per hour in dense quarters, 1½ inch in less-crowded areas, and 1 inch in the residential districts, according to the proportion of buildings and garden land. A number of old rain-water drains were found to exist, and they were supplemented by others to complete the system. The most important of the latter serve an area of 144 acres, lying within ½ mile of the harbour, but formerly draining towards the Kelani, 2½ miles away. By tunnelling under Korteboam Street, two outlets were made, discharging into the harbour, and a great improvement in the neighbourhood was apparent after this diversion had been made.

No works of a similar character had been previously attempted in Ceylon. The labour force had to be trained in the timbering of trenches and the methods required for dealing with subsoil water.

Over 40 per cent. of the length of sewers is placed below the subsoil water-level. The science of bacterial treatment of sewage had been only recently established in Europe, and its application on any but a small experimental scale was quite new to the tropics. This instalment of the scheme was handed over to the municipality for operation in September, 1910. It dealt with a net habitable area of about 600 acres, with an estimated future population of about 60,000. The cost of the works, including land-acquisition and all incidental expenses, amounted to Rs.4,848,000.

The Lake Area.—Whilst these works were in progress, the Government repeatedly urged the municipal Council to proceed with extensions of the scheme, and offered further financial assistance. The lake drainage-area was recognized as requiring attention most urgently. The construction of the main sewer from Polwatta to Layards Broadway was commenced in March 1908, and was completed in September 1909. This sewer is 2¾ miles in length; it commences with a 24-inch pipe with an invert-level of +5·86 at Polwatta, and increases in size to 4 feet 9 inches by 3 feet 2 inches at Layards Broadway. It passes under two bays of the lake, where the excavation reached a depth of −6, and cut through a bed of driftwood 2–3 feet in thickness at sea-level, which necessitated very heavy pumping. It crosses the railway by a tunnel, 23 feet below rail-level, and reaches a depth of 40 feet below the surface at the ridge between the watersheds of the river and the lake. It passes under the San Sebastian canal in three 33-inch cast-iron pipes, at a level of −10·35, and joins the 6-foot by 4-foot main sewer at −11·83 under Layards Broadway. The total cost of this sewer amounted to Rs.853,000.

In July, 1909, the Council, on the recommendation of Government, adopted a programme of work prepared by Messrs. Mansergh and Sons for the detailed drainage of the whole of the lake watershed. This programme was revised and added to during the following 2 years, until it represented an expenditure of Rs.13,800,000, dealing with a net area of 3,200 acres and an ultimate population of 193,000. The features of engineering interest included deep foundations in running sand for pumping-stations at Slave Island and Polwatta alongside the lake, two crossings under the lake at depths of 9–12 feet below water-level, and several crossings under the railway in bad ground. One of the latter was 254 feet in length, and, in order to avoid interference with the traffic, the sewer was laid through an existing flood-water culvert. The sewer trench was sunk below the foundations of the side walls of the culvert in running sand waterlogged up to the surface, and the stream was kept running through the culvert all the time during construction. Both the trench and the culvert-walls were carefully timbered, and the work was accomplished without any mishap and without causing any settlement to the railway. Many difficulties were encountered in narrow streets with rickety buildings on each side. In one place, appropriately called Ditch Lane, varying from 4 to 6 feet in width, a 12-inch sewer was laid at a depth of 28 feet below the surface through running silt.

Southern Outfall.—In November, 1911, the boundaries of the municipality were extended eastward and southward, adding a gross area of over 2,000 acres, with an estimated population of over 40,000. Besides this increase in area, the census of 1911 disclosed a growth of population which outstripped all previous records, and affected the calculations based upon previous forecasts. In order to deal with the new conditions and to provide for the possibility of still further extensions southwards, a southern outfall was suggested, with a pumping-station and treatment-works to discharge the effluent into the sea near the mouth of the Dehiwela canal; and Messrs. Mansergh and Sons were again called upon to revise the scheme for the outstanding districts. Their report, dated the 23rd January, 1913, dealt with a net area of 5,725 acres and a forecast of population estimated to the year 1951 at 373,200 within the new limits of the municipality, and provided for the possible addition as far as Mount Lavinia of a further net area of 817 acres, with an ultimate population of about 18,000, which might be included in the future. The city was divided into twenty-five districts, grouped into ten drainage-areas, with a pumping-station in each, particulars of which are given in the Appendix (p. 83). The total cost of the scheme within the new municipal boundaries was estimated at 23½ million rupees. A commission appointed to inquire into the municipal finances recommended on the 29th September, 1914, that the Government contribution towards the cost of the drainage scheme should be increased to 6 million rupees, provided the municipality undertook to complete the works to the satisfaction of the Government; that the necessary funds for that purpose should be advanced by the Government; and that, if necessary, payment of interest should be deferred. These recommendations were approved in February, 1915, and the sanctioned expenditure was increased to over 17 million rupees.

Owing to the slow progress of the house connections, further detailed drainage was postponed indefinitely, and a programme of arterial drainage alone was substituted in the uncompleted districts, the intention being to increase the number of public latrines and to establish a system of pail-tipping depots on the sewers for the disposal of night-soil, so that the pitting-ground at Narahenpitiya might be finally abolished.

The southern outfall is designed to deal ultimately with an area of 2,155 acres and a population of 65,000; the sewers and pumping-station will take six times the dry-weather flow of 25 gallons per head. Twice the dry-weather flow will be treated in sedimentation-tanks and filters, and storm-water up to four times the dry-weather flow will be dealt with in sedimentation-tanks alone. The first instalment of these works provides for a little over one-third of the ultimate quantity. The main sewer is 3¼ miles in length. It commences with a 12-inch pipe at a level of +21·60 in Kollupitiya Road, where it receives the discharge from the western pumping-station; near Havelock Town it reaches a depth of 35 feet below the surface. It crosses the Kirillapona canal with an invert level of −7·43, and finishes at Wellawatta pumping-station with cross-sectional dimensions of 3 feet 6 inches by 2 feet 4 inches, at a level of −14·62. The sewage flows through a silt-pit, with an invert level of −24·75, worked by a revolving bucket and winch. It then passes through screens with ⅝-inch spaces, which are lifted to the surface for cleaning.

The pumping-station building is 87 feet by 75 feet at ground-level. It contains a sump-floor at a level of −16·75, a pump-floor at −1·00, and an engine-floor at +12·50. It will eventually accommodate four dry-weather pumps and four storm pumps. The first instalment consists of two dry-weather centrifugal pumps with 8-inch suction and 7-inch delivery, and two storm pumps with 10-inch suction and 9-inch delivery. The rising mains are 18 inches and 24 inches in diameter respectively.

The dry-weather tanks comprise two detritus-tanks, 12 feet by 12 feet, holding 7,000 gallons each, and two double-storey sedimentation-tanks 59 feet by 26 feet, with a capacity of 3 hours flow in the sedimentation chamber, to deal with a flow of 335,000 gallons per day each. The top water-level of the tanks is at +22·50, which provides for secondary treatment of the effluent when necessitated by the increasing strength of the sewage. Storm-water tanks have not been included in the first instalment, as very little rainfull will reach the works until further detailed drainage has been introduced. The effluent channel passes under the seaside railway, and finishes with a 24-inch cast-iron pipe extending about 100 yards into the sea, with an invert-level of −11·00. A concrete channel is provided to run the sludge on to low-lying ground within the area of the treatment-works. The greater part of the main sewer, and the whole of the works at Wellawatta were excavated through running sand; and the construction of the outfall within a coffer-dam in the open sea, exposed to the full force of the south-west monsoon, presented considerable engineering difficulties. These works have now been completed, and the cost of the southern outfall, with the arterial drainage of the districts leading to it, including the western pumping-station, amounts to a little under 3 million rupees.

Sedimentation-Tanks.—Whilst the sewerage system was being continually added to, and while public latrines, and most of the large institutions, hotels, and offices were being connected, the treatment-works at Madampitiya remained unchanged, the population provided for being 60,000, with a dry-weather flow of 1½ million gallons a day. In 1913, when the flow had increased to about 2¼ million gallons, troubles began in the septic tanks. The ebullition of gases was accentuated by the tropical climate, and large masses of sludge were brought to the surface and carried over the outlet-weirs, choking the filter-beds. Owing to the excessive quantity of sewage, the storm-water tanks were used for dry-weather sewage, and these also became choked.

As a result of these experiences, the second instalment of treatment-works was commenced; and, in view of the most recent developments in sewage-treatment in Europe, it was decided to adopt the principle of the double-storey sedimentation-tanks, in which the rising gases are diverted by sloping partitions and prevented from mixing with the fresh inflowing sewage. Two radial-flow circular sedimentation-tanks were constructed during 1914 to deal with a dry-weather flow of 300,000 gallons a day each. The sewage passes through detritus-tanks, 12 feet by 12 feet by 12 feet 6 inches deep, with a capacity of about 20 minutes' flow. Three of these have been provided to serve five sedimentation-tanks. The sewage is then led through a trough to an annular inlet-channel round the centre of the tank; it overflows radially into the sedimentation-chamber, passes under a baffle, and deposits the suspended solids through a slot in the bottom of the sedimentation-chamber, and the effluent overflows at the outer circumference of the tank. The solids collect in the bottom of the tank, and the rising gases escape through the central funnel formed by the annular partitions. The sludge can be withdrawn by a stand-pipe in the centre of the tank, with its inlet at the bottom of the sludge-chamber, commanded by a sluice-valve at the outlet end, a few feet below the level of the top water. These tanks are 36 feet in diameter at the top, 23 feet in diameter at the sludge-chamber, 8 feet in diameter at the top of the central funnel, and 30 feet deep. The cost of the two tanks amounted to Rs.68,770. They effected a reduction of about 71 per cent. of the suspended solids in the sewage, and, when their efficiency had been satisfactorily established, an alteration of two of the septic tanks was put in hand. In tank No. 1, **A**-shaped frames of reinforced concrete were erected across the tank at intervals of 10 feet 3 inches throughout the whole length, the feet of the frames being fixed in the side walls of the tank and the apex standing a little above the top water. Concrete slabs were laid longitudinally upon the frames, forming a central sludge-chamber below the partitions, with a central vent about 3 feet wide at the top and two upper-storey sedimentation-channels of triangular section with narrow slots between the bottom of the slabs and the side walls for the deposit of sediment. A sludge-pipe was laid longitudinally at the bottom of the tank, with draw-off valves at intervals of 20 feet 6 inches. This tank was brought into operation in July, 1916, and the reduction of suspended solids effected by it ranged between 70 and 80 per cent. The cost of the alteration amounted to Rs.18,375.

Tank No. 2 was altered to a design by Mr. C. L. Cox, M. Inst. C.E., then the City Sanitation Engineer. Instead of placing the slabs longitudinally, he introduced a series of transverse ridge slabs similar to ridge roofing-tiles, supported on a framework at about one-third of the depth of the tank. The sedimentation took place through slots in the valleys, and the gases were diverted under the ridges to a longitudinal vent 1 foot 10 inches wide, running the full length of the tank. The reduction of suspended solids was much the same as, and occasionally a little better than, that effected by tank No. 1, but the cost of the alteration was greater, and amounted to Rs.25,570. Septic tanks Nos. 3 and 4 have since been altered in a similar manner to tank No. 1, and three more radial-flow tanks have been added, making five in all.

Wet-Weather Tanks.—Three new wet-weather tanks have been added at Madampitiya to form part of an ultimate set of fourteen. Two detritus-tanks have been provided, 22 feet by 22 feet by 11 feet deep. The sedimentation-tanks are intended to deal with a dry-weather flow of 335,000 gallons a day each, and to give an effluent good enough to turn into the river without secondary treatment. When the sewage is diluted, these tanks are intended to deal with a wet-weather flow of 2 million gallons a day each. That is to say, the rate of flow may be increased to approximately six times the dry-weather flow.

The top water-level of the tanks is at + 17·50, and the dimensions of each tank are 80 feet by 36 feet by 30 feet deep. The sedimentation-channels run the full length of the tank, but the sludge-chamber is divided transversely across the centre to form two inverted pyramids, with a sludge draw-off pipe in each. These tanks were completed in 1920, and were handed over to the sanitation engineer for operation. They produce an effluent, in the normal flow, containing only 4–6 parts per 100,000 of suspended solids. A dry-weather flow of 2 million gallons a day each was put through them as an experiment, and gave an effluent with 8–10 parts of suspended solids per 100,000; but at this rate of flow a thick scum formed, and the ebullition of gas below the scum lifted the concrete slab partitions and boiled over into the sedimentation-chambers.

Constructional Details.—Stoneware pipes were used for sewers up to 15 inches in diameter, concrete pipes were made for diameters of 18–24 inches, and mass-concrete egg-shaped sewers for larger sizes. For rain-water drains above 24 inches in diameter a segmental design was adopted, with curved invert and arch and vertical side walls. This form adapted itself to situations where very little headway was available, was economical in shuttering, and saved concrete under the haunches which would have been necessary as filling between a circular section and the bottom of the trench in bad ground. The pumping-stations were constructed so that no sewage came in contact with the atmosphere inside the buildings. Large ventilating-towers over the incoming sewers carried the gases above the roof-level. The buildings were specially designed for a tropical climate, with ample height of roof, broad projecting eaves, and verandas arranged to exclude direct sunlight during the heat of the day; and large latticed openings were provided under the eaves, sheltered from the rain, but affording thorough ventilation in the engine-house.

The works up-to-date have cost 18 million rupees. Sixty-one per cent. of the habitable area of the city is provided with drainage in every street, 28 per cent. with arterial drainage only, and 11 per cent. remains to be dealt with. One hundred and twenty-eight miles of sewers and rain-water drains have been constructed. The dry-weather flow averages now about 4½ million gallons a day. In connection with the sewers, the municipal council has constructed thirty-six public latrines and thirteen pail-tipping depots, and more are to be added. The first latrine upon the water-carriage system was established in 1910, and was used by about 7,000 persons daily. The public latrines now are used daily by more than one-third of the entire population. Night-soil from premises not connected with sewers is collected into pails and carried to the tipping-depots. These consist of a hopper or trough, where the contents of the pails are washed by a hose into the sewer.

Since the first instalment of the drainage-system was brought into general use in 1911, the average death-rate has fallen from 33 to 27 per thousand, a reduction of 18 per cent.; and it is estimated that a saving of a million rupees annually has been effected in funeral expenses alone, and a vast deal more in wages saved by reduction of sickness. Colombo is now believed to be the healthiest city in the tropics.

The Author was resident engineer from 1902 to 1912, and again in 1920 and 1921. Mr. M. R. Atkins, M. Inst. C.E., was in charge from 1912 to 1919, and the late Mr. R. G. Waterhouse in 1919 and 1920. The Author expresses his thanks to Messrs. Mansergh and Sons for their kind assistance in the preparation of the Paper. The Appendix gives particulars of the pumping-stations and drainage-areas.

The Paper is accompanied by one map, from which Plate 1 has been prepared, and by the following Appendix.

APPENDIX.

PUMPING-STATIONS.

Station.	Population 1951.	Watershed.	Lift.	B.HP. Total.	Power.
		Acres.	Feet.		
Northern	326,000	4,387	46	409	Suction gas.
Harbour	54,520	666	21	82	,, ,,
Slave Island	18,335	193	23	53	,, ,,
Polwatta	22,690	635	19	68	,, ,,
Eastern	17,465	499	22	84	,, ,,
Vystwyke	8,800	110	26	23	Crude oil.
Maligawatta	30,170	431	22	78	Suction gas.
Timbirigasyaya[1] . . .	7,225	289	28	10	
Western	5,360	134	18	23	Crude oil.
Southern	47,200	1,338	38	175	Suction gas.

[1] A temporary pumping-station has been established at Jawatta, dealing with a portion of this area.

(*Paper No. 4390.*)

"Sewage-Disposal in South Africa, with Special Reference to Sludge-Treatment at Pretoria."

By Frank Walton Jameson.

Recent evolution in sewage-disposal may be conveniently divided into three stages:—(1) The Cameron–Exeter septic-tank process; (2) the Emscher–Imhoff and Travis process; (3) the process applied in Birmingham by Mr. J. D. Watson, M. Inst. C.E. The first system depended on the introduction of a septic or liquefaction tank with primary or secondary oxidizing-beds and subsequent land treatment. Though the fantastic claims made by ill-informed persons regarding the ability of this tank to decompose all solids and to produce by anaerobic action a clarified liquor and an infinitesimal deposit in the form of humus were not borne out, the possibility of breaking down solids by bio-chemical means was demonstrated. The second stage appears to be confined largely to experimental and practical research on tank-effluent and sludge treatment. At this stage the earlier rectangular septic tank was replaced by the cylindrical type of tank developed in Germany, which, instead of producing an elongated horizontal flow, provided both vertical and horizontal movement of water. These tanks were designed to allow sewage, during its passage through the tank, to deposit suspended matter on a horizontal floor, and to permit such matter to rest there as sludge for a period of time, the partially clarified liquor passing horizontally through the tank at the surface. They were, in fact, combined sedimentation and septic tanks. Various mechanical contrivances were introduced for the purpose of sludging the lower pit of the tank and drawing off all supernatant liquor.

While the ability of such tanks to separate solids from liquids was established, much remained yet to be done. It became evident that the lower or sedimentation content of the tank produced reactionary results in the upper liquid content of the tank, together with considerable quantities of gas, for septic action rapidly took place in the lower sludge content. The third evolutionary stage may be regarded as synchronizing with Messrs. Watson's and O'Shaughnessy's invaluable work at Birmingham. Using as their basis the Dortmund circular and cone tank, they improved upon the German design. An inverted pipe was carried 10 feet below the surface of the liquor content of the tank. This tank does not appear to have been used by Mr. Watson for sweet-sewage settlement, but as a secondary sedimentation-tank for liquor from first treatment in a rectangular tank. Mr. Watson discarded the dual chamber used by Messrs. Travis and Imhoff, and used separate compartments. The removal of organic matter in the bottom zone of the cone-shaped tank was effected either by providing vertical and horizontal pipe legs as sludge-outlets, or by utilizing a 4-foot static head for ejecting the sludge contents, or by pumping out the contents where a head was not available. It was clear to the Author from a study of this tank design that, if used for primary treatment of sewage, a step towards a fourth stage of evolution might be reached for South African practice. The function of the Watson tank as adopted in Pretoria is the reverse of its use at Birmingham, for it deals with crude sewage, not settled sewage, and in this sphere it is highly efficient.

The advantages of the Watson tank as a medium of primary separation of organic matter from crude sewage were accepted by the Author as applying the Hampton or Travis theory. It was argued that, if this theory were sound, crude sewage might be primarily separated from organic matter as early, not as late, as possible, and before it became septic. The use of the Watson tank as a primary separator ensures a flow of sweet sewage to the oxidizing-beds and of sludge to an entirely separate septic tank. The Author is definitely of opinion that the dual and combined compartment for sewage tankage treatment, as in the Emscher-Imhoff and Travis designs, will not become standard South African practice, though it may be more suited to temperate and wetter climates. In 1919 Mr. Watson, referring to the Emscher-Imhoff tank, made the following statements[1] :—

> "With the idea of carrying on the functions of oxidizing the liquid of sewage and the septicization of sludge simultaneously, Dr. Imhoff patented about 1904 . . . a two-storey tank, which had the Travis hydrolytic tank for its prototype. The lower chamber of this tank

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[1] Proceedings of the Institution of Municipal and County Engineers, vol. xlvi, pp. 201 and 207.

is for fermentation, and the upper one for sedimentation. The Birmingham plan provides two separate shallow tanks, one placed alongside the other, to do the same work . . . Imhoff found that he required his fermentation chambers to be large enough to contain from three to six months' storage of sludge."

PRETORIA SEWAGE SCHEME.

The system was designed by the Author 12 years ago, and has been in operation for 10 years. The population in the sewered area is approximately 30,000, and 10,000 are dealt with by pail removal. The whole contents of both systems flows to the outfall works. The scheme is that known as the separate system, the outfall sewer being designed to take storm-water equal to the dry-weather flow. The average rainfall is 30 inches per annum. The intercepting-trap between the public sewer and private drains was discarded, and the system of using each private connection as a public sewer ventilator outlet was adopted. There have been only one or two cases, known to the Author, of nuisance arising from sewer-gas throughout the whole sewered area, and in these cases the origin was found to be temporary stagnation in sewers of large diameter, whilst being connected to adjoining properties. Experience of the Pretoria system shows that a well-laid and well-graded sewer should produce practically no gas. There are only two public sewer ventilators in the whole public reticulation of 60 miles.

The public authority of Pretoria agreed in the early stages of management to appoint Dr. J. J. Boyd, the Medical Officer of Health, conjointly with the Author, to the control of the outfall works, although the Council strongly opposed such proposals. The Author insisted that thoroughly scientific handling of such works could only be attained by close co-operation between the engineer and the bio-chemist. It was further agreed that an experienced professional man, possessing a knowledge of bio-chemistry, should be placed in direct charge of the works, as sewage-works manager. This officer works under the joint jurisdiction of the Engineer and the Medical Officer of Health.

Description of Outfall Works.—The outfall works consist of a screening-chamber for heavier matter such as rags, paper, etc., grit-channels, Watson-Dortmund tanks, gauging-weirs, dosing-tanks, circular oxidizing-beds fed by revolving sprinklers (the oxidizing-beds having a single continuous flow), and rectangular septic tanks into which sludge is discharged from the primary Watson tanks. The effluent from the oxidizing-beds passes directly into the Aapies stream immediately below the works. The daily dry-weather flow of sewage is 1½ million gallons; the original dry-weather flow of the Aapies stream was 1½ million gallons, and it is to-day 4½ million gallons. The Aapies stream passes through a fairly thickly populated semi-urban district for 5 miles below the outfall works. Approximately 7½ acres of heavy loam fallow soil is at present being used for the dry-weather treatment of septic effluent. Approximately a further 2½ acres is required during the hot weather. Sludge, after removal from the septic tank, is dried atmospherically during the 8-month dry-weather period. It is thereafter crushed into a fine powder, mixed with phosphates, and sold as a fertilizer. The standard of purification of the effluent leaving the oxidizing-beds and passing directly into the stream is that laid down in the Eighth Report of the Royal Commission, and there has been no difficulty in conforming to its requirements. The standard of effluent requires:—

(*a*) That it should contain not more than 3 parts of suspended matter per 100,000.

(*b*) That it should not take up more than 2 parts of dissolved oxygen per 100,000 in 5 days at 65° F.

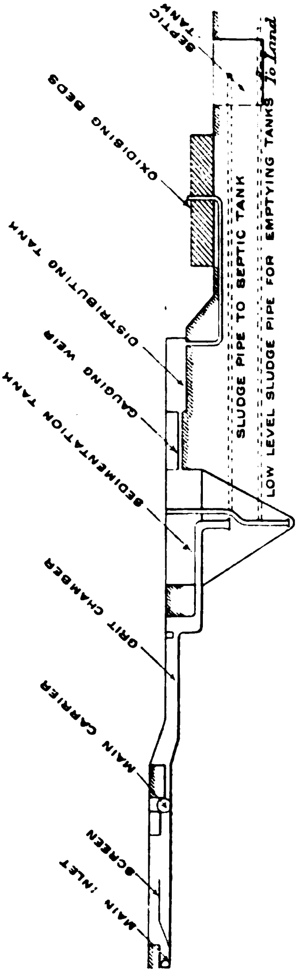

SEPTIC TANK
OXIDISING BEDS
DISTRIBUTING TANK
GAUGING WEIR
SEDIMENTATION TANK
GRIT CHAMBER
MAIN CARRIER
SCREEN
MAIN INLET
SLUDGE PIPE TO SEPTIC TANK
LOW LEVEL SLUDGE PIPE FOR EMPTYING TANKS *To Land*

Fig. 1.

ARRANGEMENT OF WORKS AT PRETORIA.

The unsewered area is dealt with by pail removal, and the pail contents are emptied into a collecting-tank. The following is the analysis of sewage content in the collecting-tank before entering the main outfall sewer (p. 88).

	Parts per 100,000.
(1) Suspended solids	225·6
(2) Free and saline ammonia as nitrogen	36·8
Albuminoid „ „ „	8·4
Nitrous nitrogen	Nil
Nitric „	Nil
Chlorine	20·8
Oxygen absorbed from (1) in 3 minutes	19·6
$\frac{N}{80}$ permanganate at 80° F. (2) in 4 hours	47·4
Dissolved oxygen absorbed in 5 days at 65° F.	136·0

The analysis of the sewage after contact and dilution with the sewage in the main outfall sewer is :—

	Parts per 100,000.
(1) Suspended solids	77·2
(2) Free and saline ammonia as nitrogen	16·8
Albuminoid „ „ „	3·4
Nitrous nitrogen	Nil
Nitric „	Nil
Chlorine	9·6
Oxygen absorbed from (1) in 3 minutes	7·8
$\frac{N}{80}$ permanganate at 80° F. (2) in 4 hours	18·0
Dissolved oxygen absorbed in 5 days at 65° F.	46·0

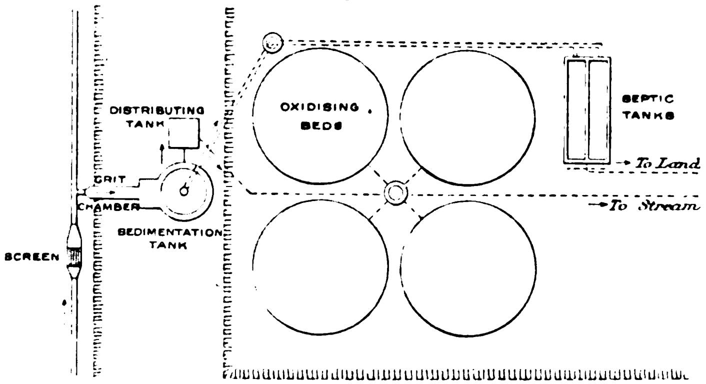
The partly septic contents of the pails, when diluted and after contact with the town sewage in the main sewer, is considerably purified; in fact, it does not differ materially from the figures given below. This is accounted for by the fact that the diluting sewage in the main sewer at this period, 2 a.m., is almost pure water from flushing-cisterns. The analysis of sewage (dry-weather flow) in the main outfall sewer, taken after the pail-content inflow had ceased, is as follows :—

	Parts per 100,000.
(1) Suspended solids	28·2
(2) Free and saline ammonia as nitrogen	3·0
Albuminoid „ „ „	1·0
Nitrous nitrogen	Nil
Nitric „	Nil
Chlorine	5·6
Oxygen absorbed from (1) in 3 minutes	2·08
$\frac{N}{80}$ permanganate at 80° F. (2) in 4 hours	6·13
Dissolved oxygen absorbed in 5 days at 65° F.	31·0

The quantity of water used in dilution of the stercus content is 54,000 gallons per diem, covering a period of 3 hours at night; this represents 20 gallons per pail.

Figs. 1 and *2* show diagrammatically the lay-out of one of the four compartments of the works. The main outfall sewer is of reinforced concrete. The works are divided into four compartments, each compartment being complete in itself and dealing with approximately 333,000 gallons per diem. Sedimentation-tanks have a capacity of one-sixth of the total daily volume entering each compartment. The oxidizing-beds deal with 150 gallons per square yard per diem, and are each of 560 square yards area. The beds are 6 feet deep, the media being broken granite resting on a false floor of half-round tiles which in turn rest on a concrete floor. Each of the septic tanks has a capacity of approximately one quarter of the total daily volume entering each compartment. The period taken to fill No. 1 tank with sludge was approximately 12 months; thereafter a period of rest was given to No. 1 tank of 12 months until most of the sludge was comparatively dry. No. 2 tank was filling while No. 1 was resting. The sludge from the Watson tank is of average quality, and approximately 96 per cent. of its weight on leaving the sedimentation-tanks for the septic tanks is represented by moisture. The ejection occurs at 7 a.m. and 3 p.m., when the maximum suspended matter is collected. The total volume of sludge and liquor which is discharged from the sedimentation-tanks to the septic tanks is 16,000 gallons per diem, representing approximately 1 per cent. of the total volume of sewage entering the sedimentation-tanks. Sixty per cent. of the total contents entering the septic tank is retained therein at each operation. The septic effluent under this method was passed twice daily from the septic tanks on to fallow land by gravitation. Under no circumstances is this liquor allowed to pass into the Aapies

Fig. 2.



Plan of Arrangement of One of the Four Compartments of the Works.

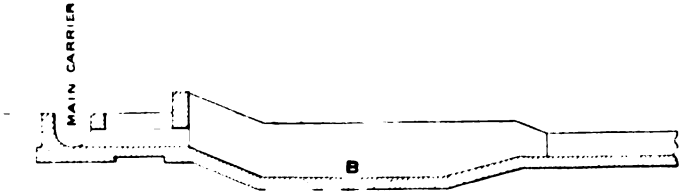
stream. The sludge produced from 1½ million gallons of sewage per diem is approximately one dry ton. The crops grown on the cultivated farm area of 25 acres are lucerne, teff, and barley. Cultivation of the land is conducted as follows:—

The land is ploughed and thoroughly harrowed, cross ploughed, and again thoroughly harrowed to eradicate weeds. Teff grass is sown in the spring, and the hay therefrom is baled for winter food. The variety of lucerne found most suitable is "Hunter's River." This is sown with a catch crop of barley in the autumn, and the barley is cut off during the winter. The lucerne of the spring is well established and more than holds its own against weeds in summer. An average of ten cuttings per annum of lucerne is obtained. This is either sold green or dried and baled.

EXPERIENCE GAINED DURING 9 YEARS' OPERATION OF THE WORKS.

The foregoing data should be taken as an index of the earlier position of the scheme as regards septic-tank treatment. Later experience and scientific investigation enabled further valuable data to be recorded. One of the earliest difficulties occurred in connection with the grit-chamber, which has been adopted from English practice. This chamber was initially as shown in *Fig. 3.* It was found that septic action took place in the chamber as sludge accumulated at this point, although in temperate climates no trouble would arise. In order to ascertain an invert line best suited to overcome septic action and still to trap grit, a wooden false floor was fixed in the chamber B, covering its whole area. This floor was raised to different angles with the hope of retaining a sump to serve as a grit-chamber and avoiding septic action, but without success. Finally the floor was laid dead level. Sewage deposited much of its grit content on the invert of the chamber, which was

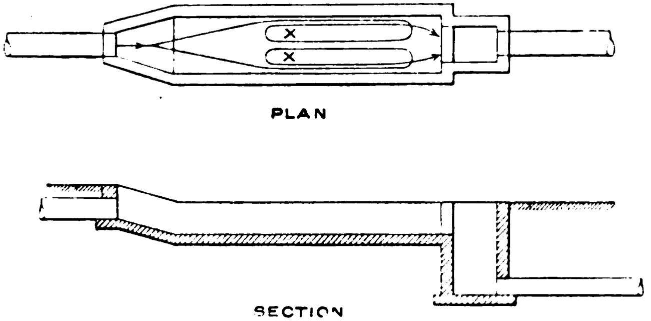
Fig. 3.



OBSOLETE TYPE OF GRIT-CHAMBER.

27 feet long and 6 feet wide, in the direction indicated in *Figs. 4.* Grit was carried by currents to the centre of the channel and deposited on the lines X X. The water in this zone is practically quiescent during dry-weather flow. Sewage robbed of heavier grit passes from this channel to the Watson sedimentation-tank. The sedimentation-tank will cause trouble from blocking by grit unless a grit-channel and chamber are provided. The normal depth of water in the grit-channel is 4 inches. This shallow and open compartment permits of grit being easily removed from time to time with hand shovels. Dry-weather grit content from all the channels is removed daily by one native in 2 hours. The time taken during wet weather is 3 hours daily. The sewage passes from the grit-chamber and enters the Watson sedimentation-tank, which is circular for a depth of 5 feet and cone-shaped to a depth of 25 feet. Sludging is performed by opening a valve fixed to a horizontal outlet pipe, outside the tank. This pipe is attached to a vertical pipe inside the Watson tank. When the valve is opened, the static head in the tank forces the sludge from the bottom of the tank through the vertical leg and thence to the septic tank (*Fig. 2*). The sludging period cannot be fixed arbitrarily for all systems. During warm weather, sludging must be carried out more frequently than in cold weather, for septic action occurs readily in the lower zones of the sludge, affecting to a marked degree the sweeter sewage on the surface. It has been found that, if sludging is postponed for 24 hours, sludge rises to the surface, causing a marked reaction in the aerobic state of the surface liquor of the tank.

Figs. 4.



NEW TYPE OF GRIT-CHAMBER.

The preference of some sewage engineers for the Emscher–Imhoff or Travis type of tank, as opposed to the Watson, has always perplexed the Author. It is argued by some authorities who use the

Emscher–Imhoff tank that, if the lower or fermentation contents of the tank is not frequently sludged out (Imhoff says it should not be), during this undisturbed period the sludge content will liquefy without affecting septically the higher zones of the liquor in such a tank. But that is not the Author's experience, and it is certainly a dangerous experiment to try in South Africa. Again it is difficult to understand how septic sludge, if allowed to remain in the fermentation-tank for one or more months, can be ejected through pipes under a static head. The Author's experience is that the sludge content of such a tank very early in its life cycle becomes massed into a cohesive medium, most of which defies any removal by such methods. Efficient design requires early discharge of the sludge, not its retention in the sedimentation-tank. With the Cameron septic tank it was accepted as sound practice to pass septic liquor directly on to filter-beds; but later scientific investigations proved that effluent rendered putrescent by contact with anaerobic flora in the septic tank carried with it also quantities of organic matter which, if passed on to oxidizing- or filter-beds, retarded or defeated the desired results. Unfortunately, this lesson had not been fully learned in South Africa.

The supernatant liquor from the Watson tank is collected in the dosing-tank, which is also used as a grease-trap: quantities of grease and fat would otherwise pass therefrom to the surface medium of the oxidizing-beds, and thus reduce the efficiency of these beds by clogging. Analysis of Pretoria water-supply taken from the mains is as follows:—

	Parts per 100,000
Calcium carbonate	10·75
Magnesium "	6·72
Sodium chloride	0·288
Loss on ignition	2·0

Grease and fat in water of this nature produces a scum of fatty matter in the dosing-tank. To prevent much of the grease passing from the chamber a wood baffler is fixed in the tank from 6 inches above the surface of the liquor to within 1 inch of the floor. The fat and grease adhere to the walls of the tank in this divided compartment and are easily removed with a scraper. Since the introduction of this trap no difficulty from fatty matter has been experienced with the oxidizing-beds. A gauging-weir is fixed between the outlet of the Watson tank and the inlet of the dosing-tank. Its function is to operate a float working in a separate annexe chamber. The float is connected to the usual drum clock, the sewage flow being mechanically recorded by a cam on the cylinder.

The automatic record, in gallons per hour, enables the manager to check the rate of daily or hourly flow of sewage passing to each set of oxidizing-beds. The effluent from this compartment passes automatically, under the control of a balanced float and valve, to the oxidizing-beds, thus securing an equal dosing to each bed. The oxidizing-beds are designed on a conservative basis of sewage-treatment of 150 gallons per square yard per diem, with a view to produce an effluent which will meet the standard requirements, particularly having regard to the nature of the semi-urban and riparian requirements of the Aapies stream below the works.

The analysis of supernatant sewage from the Watson tank and dosing-tank before entering the oxidizing-beds is as follows:—

	Parts per 100,000.
(1) Suspended solids	14·2
(2) Free and saline ammonia as nitrogen	3·25
Albuminoid ,, ,, ,,	0·88
Nitrous nitrogen	Nil
Nitric ,,	Nil
Chlorine	5·3
Oxygen absorbed from (1) in 3 minutes	1·9
$\frac{N}{80}$ permanganate at 80° F. (2) in 4 hours	5·1
Dissolved oxygen absorbed in 5 days at 60° F.	22·0

The analysis of sewage-effluent after passing through the oxidizing-beds is:—

	Parts per 100,000.
(1) Suspended solids	2·60
(2) Free and saline ammonia as nitrogen	1·80
Albuminoid ,, ,, ,,	0·30
Nitric Nitrogen	2·05
Nitrous ,,	0·15
Chlorine	5·90
Oxygen absorbed from (1) in 3 minutes	0·56
$\frac{N}{80}$ Permanganate at 80° F. (2) in 4 hours	1·40
Dissolved oxygen absorbed in 5 days at 65° F.	2·20

Theoretically each molecule of liquid sewage carries with it atmospheric oxygen and feeds the beds with some of the oxygen required for bacterial and chemical action. Below the tiled floor of the oxidizing-bed an air buffer or zone exists between the medium and the concrete floor. During intervals when the bed is at rest (during the collection of clarified sewage in the dosing-tank), atmospheric oxygen tends to be absorbed vertically into the lower zones of the oxidizing media, filling the voids temporarily freed from liquid. It is true that colloidal matter or humus is deposited on the concrete floor of the collecting-channels of the oxidizing-bed, but such colloid matter can be readily removed after reaching a bulk inimical to the functional response of self-cleansing flow of the effluent channels. The process is as follows:—The effluent outlet to the stream is temporarily cut off, the floor is thoroughly hosed with water, and the colloid content of the channels is passed on to the land. The work of the oxidizing-beds need not be suspended in the meantime. This operation only requires to be carried out about once in 12 months, and occupies about 1 hour per bed. The colloid matter is rapidly digested by disposal on land. It is quite unnecessary to introduce colloid or other humus tanks for final settlement before passing the effluent to the stream, which is common practice in Europe. The Author believes that an almost indefinite life may be ascribed to oxidizing-beds for average domestic sewage by using one in a set of four as a stand-by. Analytical observation of the work done by these beds enables the manager to fix a period when one or more may need rest. When such a period is reached, the bed chosen for rest is allowed to remain undisturbed for a dry-weather period of 6 weeks to 2 months. The bed is then completely dry. If suspended matter has disorganized the bacterial life-cycle, and if it is desired to cleanse the bed of humus, in the dry atmosphere of the Transvaal this is easily done by dosing the dry medium of the bed after rest with effluent, thus driving out the humus content ahead of the flush. The beds can then be used again for sewage, and, by gradually increasing the doses, the bed soon regains its bacterial flora and efficiency. Medium, needing more drastic treatment, should be removed after permitting the bed to dry out, and then be passed over a stone grading-screen by an endless elevated cup-chain. This process will regrade the stone and ultimately shake free the dry humus, leaving the stone medium perfectly clean for return to the bed without washing. This is more economical than washing the medium. (Mr. Watson gave the pre-war cost of washing at Birmingham as 1*s*. 6*d*. per cubic yard.)

The Effect of Stercus Deposit.—The decision to use the outfall works to receive an effluent, which the bio-chemist declared already septic on reaching the stercus-tank, was not arrived at without due regard to its effect upon the sweet sewage at the outfall works. It was indeed by recognition of the danger involved that success was ultimately attained. The economic factor exerted pressure on the Council's officers. A saving of £3,700 per annum would be effected if the cost of transport and burial of stercus from the unsewered area could be eliminated. A tank, therefore, was designed, the stercus contents were discharged therein, and the call for economy was appeased. Within 12 months of this superdosing with night-soil content covering a 3-hour period, the oxidizing-beds lost the efficiency they had established prior to such dosing. The percentage of matter in suspension rose, and the nitrates fell. The Author deeply regrets his inability to give the analysis, owing to the untimely ill-health and death of his valued co-worker, Mr. Roger Lees, who was the manager of the outfall works. During this period the quantities of pail stercus content and sewage were about 40,000 and 80,000 gallons respectively.

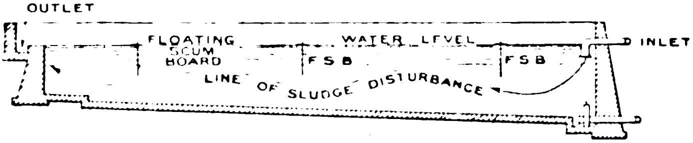
Observation of stercus content showed that—(*a*) decomposition or septic action was produced before reaching the stercus-tank, due to 1, 2, or 3 days' delay in pail removal (pail-removal services are now greatly improved), and (*b*) deposition of fly-egg, and partial incubation of pupæ occurred during this period. These difficulties were dealt with as follows. It was first determined that the cycle of stercus flow could be arbitrarily fixed. The stercus discharge period occurred regularly and occupied approximately 3 hours between the hours of 2 a.m. and 5 a.m., the period at which the flow of sewage from the reticulated area was at its minimum. It was decided that during this recorded and charted definite period of flow the sewage reaching the works should pass to the sedimentation-tanks, and thence directly for irrigation disposal on land—the worst soil at the outfall works being selected for such purpose. Within 15 months of this diversion of flow of stercus effluent from the oxidizing-beds the latter regained their former satisfactory efficiency, and the analysis of effluent therefrom once again conformed with the required standard. The 7 a.m. draw-off from the sedimentation-tank robbed it of any septic reaction set up in the 3-hour contact. No difficulty arose from flies except during the warm weather, when crawling flies covered the surface of the septic-tank sludge. A mixture of arsenite of soda and sugar in the proportion of 1 lb. of arsenite of soda to 10 lbs. of treacle, mixed with 10 gallons of water, is sprayed daily on the dry sludge surface and the walls and coping of the tank; this acts fairly efficiently. With the increase of sewage in the main outfall the contamination due to night-soil has vanished, and the sewage combined with night-soil is treated as ordinary sewage.

THE EVOLUTION OF THE SEPTIC TANK AND SLUDGE-TREATMENT.

For 8 months of the year throughout most of the Union of South Africa except in the Eastern Transvaal and Natal, there is a dry-weather period. Full value should be taken of climate and land during this dry period in dealing with both sludge and effluent. The sludge problem of to-day concerns the so-called septic tanks, and there is hope of a future solution in the utilization of the sludge products of the tank, either directly on the land in dry weather, or by drying and storage.

It has been shown that sludge from sedimentation-tanks is brought to the septic-tank chamber twice in the 24-hour cycle, and that the septic tank is used as a sludge-digestion chamber. The present methods in use were evolved by experience with various systems.

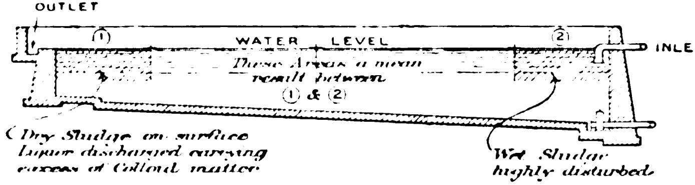
Fig. 5.


OUTLET
FLOATING SCUM BOARD
WATER LEVEL
F S B
F S B
INLET
LINE OF SLUDGE DISTURBANCE

Original Type of Septic Tank.

First System Used at Pretoria.—The original design was that of the open type, shown in *Fig. 5.* (The line of sludge-disturbance is exaggerated.) Within a short period it was clear that the floating scum-boards were acting detrimentally to the efficiency of the tank. The sludge content of the tank was hopelessly disturbed during each inflow from the sedimentation-tank even when inlet and outlet channels were used in place of pipes, and the scum-boards impeded the natural flow of the septic effluent. There was marked agitation of the tank contents at point (2) in *Fig. 6.* The rocking movement set up in the sludge surface at this point was graduated over the inner compartments. It was evident that such design would not meet the prevailing conditions of high and dry temperature and inlet under static head. It was found that the life of a septic tank without baffles, with a liquid capacity equal to one-quarter of the daily flow entering each set of works, appeared to be 12 months.

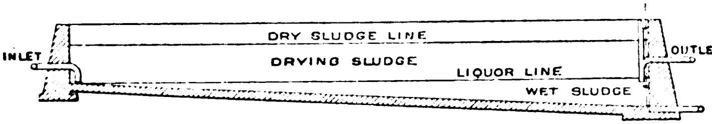
Fig. 6.


OUTLET
(1)
WATER LEVEL
(2)
INLET
These Areas a mean result between
(1) & (2)
Dry Sludge on surface Liquor discharged carrying excess of Colloid matter
Wet Sludge highly disturbed

Original Type of Septic Tank.

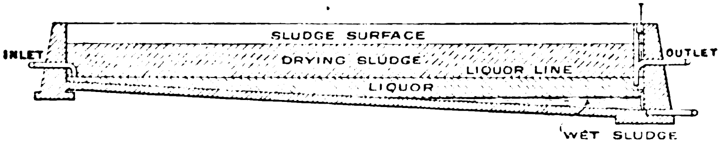
Second System.—The second design of the tank is shown in *Fig. 7.* The inlets and outlets were reversed—the shallow end became the inlet and the deep end the outlet. The collecting-channel was filled in and abandoned. When the tank was empty except for a small deposit of ripened sludge, fresh sludge was introduced from the Watson tank. Sludge accumulated until it reached the outlet-level. "Dry sludge" then formed above this line, and "wet sludge" below. The tank has a life cycle of approximately 12 months under the process of intermittent filling and discharge, and its contents are allowed to rest and dry for another 12 months, making 24 months before emptying. During the second 12 months slight fermentation is exhibited, and a certain quantity of liquor is produced and drawn off to the land. A cross section of the tank after 24 months shows dry sludge about 3 feet deep, the remainder being wet sludge, which is highly offensive. During the period when this arrangement was in use it was discovered that dry sludge was a fertilizer of some value. The desideratum then became a tank that would produce as much dry sludge as possible.

Fig. 7.



New Type of Septic Tank.

Fig. 8.

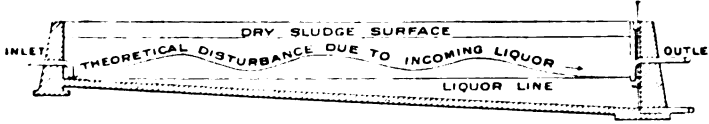


New Type of Septic Tank.

Third System.—Experiments therefore were started to obtain evidence as to how best to accomplish this purpose. A movable outlet-pipe was arranged in the septic tank, attached to which, and inside the tank, was a vertical inspection-pipe. The outlet-pipe was made to follow the liquor line of the tank. As the liquor dropped, so did the pipe, until ultimately the lowest or liquor line of the tank at the end of its life cycle was ascertained. The liquor line at the end of 12 months was found to be a narrow margin between the dry and wet sludge. At the end of 12 months the tank was full as in the first system, but it contained a greater quantity of dry sludge.

A section of the tank at the end of the filling for the 12 months period is shown in *Fig. 7.* The state of the "dried out" tank at the end of the 12 months period is shown in *Fig. 8.* The offensive condition of the contents of wet sludge largely diminished. There were, however, still difficulties to be overcome. *Fig. 9* shows an exaggerated wave of incoming liquor. This agitation occurred after introducing inlet and outlet pipes in place of a pipe inlet and collecting-channel drawing off at surface level. The outlet shown in *Fig. 9* was kept shut during filling from the Watson tank. The movement of incoming sludge from the Watson tank disturbed and broke the growth of cohesive drying sludge above the outlet level. This method permits partially unripened and offensive effluent containing organic matter to leave the outlet when it should still remain in the tank.

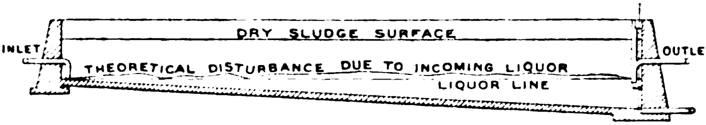
Fig. 9.


DRY SLUDGE SURFACE
INLET
THEORETICAL DISTURBANCE DUE TO INCOMING LIQUOR
OUTLET
LIQUOR LINE

OUTLET CLOSED.

Fourth System.—The fourth system is shown in *Fig. 10.* The outlet was kept open during the filling period. No material disturbance then occurred in the upper drying sludge layer. The rise and fall of the dry sludge due to the incoming wet sludge was no longer a disturbing factor when the outlet was temporarily shut. The removal of septic liquor from the septic tank as often as it would flow away improved the tank as a drying-chamber. A fifth stage of tank treatment with the outlet left permanently open is being tried. Operations are carried on in such a manner as to allow liquor to flow uninterruptedly from the septic tank. The liquor discharged is inoffensive and inodorous.

Fig. 10.


DRY SLUDGE SURFACE
INLET
OUTLET
THEORETICAL DISTURBANCE DUE TO INCOMING LIQUOR
LIQUOR LINE

OUTLET OPEN.

Fifth System.—The sludge dries more rapidly in the upper layers of the tank, and, as it is formed, it is removed to make room for fresh layers to be built up. How long this process will go forward without the necessity for emptying and removal of the entire tank contents remains to be determined. The object is to dry sludge as rapidly as possible, not to keep it wet as in double-storied tanks. As far as the Author is able to judge, sludge removed under double-storied methods in temperate climates persistently resists the extraction of moisture under a natural atmospheric drying process, covering an impracticable time-limit of exposure; while even in the case of activated sludge only 80 per cent. of the water content is extracted by centrifugal machines. In the case of the Pretoria tank, receiving ejected sludge containing 96 per cent. of moisture, sludge containing only 76 per cent. of water is obtained from the septic tank in the upper layers. This sludge is inodorous, and is formed rapidly during dry weather. During the dry season it is removed as it forms in the tank and spread on the surface of the ground to a depth of 12 inches, and within 1 or 2 months it is dry enough to be collected and ground into a fine powder. It is then mixed with a proportion of phosphates and used as a fertilizer. The following is an analysis of the dried sludge combined with phosphates as fertilizers :—

Powdered Sludge.

Total nitrogen.	3·16 per cent.
Total phosphoric oxide	5·03 ,,

Sludge Powder and Phosphates (as crushed bone).

Nitrogen	5·5 per cent.
Phosphoric oxide	8·5 ,,

The Paper would be incomplete if it did not record disappointments. During the war there was difficulty in obtaining information as to the progress of investigations overseas; and the useful work being done in Birmingham by Messrs. Watson and O'Shaughnessy, and the progress taking place in the United States of America, were at that time unknown to the Author. Later it was found that Mr. Watson was endeavouring to dry sludge artificially and use it as a fertilizer. The Author exchanged notes with him on the subject. The work of investigation meantime was carried on in Pretoria by Mr. Roger Lees, the Works Manager. It was at this stage, working in co-operation with Dr. Juritz, the Chief Agricultural Research Chemist of the Union Government at Cape Town, that it was decided to add phosphates to the sludge, but such products were difficult to procure during the war. The Author, however, constructed a small temporary steam plant capable of digesting offal and waste—oxen, horses, donkeys, dogs, etc., which were collected in the city instead of being buried. Phosphates were then available in certain limited quantities for mixing with sludge, the proportions used being approximately 50 per cent. of powdered sludge to 50 per cent. of powdered meat and bone.

Further experiments are being undertaken in the direction of drying sludge from the Watson tank on deposits of household rubbish without passing it to septic tanks. The Author hopes that the ultimate use of the septic tank may be confined to the wet season only. The sludge passed on to household rubbish dries very rapidly, and may be collected with or free from rubbish and ground into a powder for use on farm soil as a tillage and fertilizing medium.

The Author wishes to record the assistance rendered to him in the construction and working of the Pretoria sewage-scheme by Dr. J. J. Boyd, Medical Officer of Health, Pretoria Municipality; Mr. H. Conyers-Kirby, now Government Inspector, Cape Provincial Government; Mr. G. Storrar, now Town Engineer, Pretoria; Mr. M. J. Lundie, the Manager of the sewage outfall works; and particularly by the late Mr. W. Roger Lees, who, in spite of ill-health, carried out much of the investigation described.

The Paper is accompanied by thirteen prints, from some of which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

ANALYSES.

Combined Sludge and House Refuse (50 per Cent. of Each).

Nitrogen	2·14 per cent.
Potash (K_2O)	0·11 „
Phosphoric oxide (P_2O_5)	3·13 „
Lime (CaO)	1·85 „

House Refuse.

Nitrogen	1·13 per cent.
Potash (K_2O)	0·04 „
Phosphoric oxide (P_2O_5)	1·23 „
Lime (CaO)	1·42 „

Dry Sludge.

Nitrogen	3·16 per cent.
Potash (K_2O)	0·19 „
Phosphoric oxide (P_2O_5)	5·03 „
Lime (CaO)	2·28 „

Sheep Manure.

Nitrogen	Potash	Phosphoric oxide
0·56 per cent.	0·15 per cent.	0·31 per cent.

Sheep Urine.

Nitrogen	Potash	Phosphoric oxide
1·95 per cent.	2·26 per cent.	0·01 per cent.

Fowl Manure.

Nitrogen	Potash	Phosphoric oxide
1·6 per cent.	0·8 per cent.	1·7 per cent.

Combined Sludge Powder and Phosphates.

(as crushed bone)

Nitrogen	5·5 per cent.
Phosphoric oxide	8·5 „

(*Paper No. 4459.*)

"Main Drainage of the Southern Suburbs of the City of Capetown, South Africa."

By DAVID ERNEST LLOYD-DAVIES, M. Inst. C.E.

THE district served by the main-drainage scheme comprises the former municipalities of Woodstock, Maitland, Mowbray, Rondebosch, and Claremont (*Fig. 1*). It covers an aggregate area of 30 square miles. The extreme length measured from the nothern boundary of Maitland to the southern limit of Claremont is 8 miles, and the greatest width, that across Claremont, is 6 miles. Approximately 15 square miles are built upon and sewered by the scheme with which this Paper deals. Generally speaking, the land falls evenly from the mountains towards the sea, but there are large tracts of land around Table Bay lying little above sea-level, portions of which are subject to flooding in winter. The general configuration of the suburbs precluded the possibility of a gravitation scheme, and the prevailing currents in the bay were found, after investigation, to be unfavourable for sea outfalls, such as Capetown proper and Sea Point possess.

Description of the Scheme.—The five main intercepting-sewers follow the natural valley lines of the area to concentrate at the main pumping-station at Raapenberg (A, *Fig. 1*). The sewage from the outlying low-level districts, which have not yet been completed, will be lifted from the sub-pumping-stations at points C, D, E, F, G, and H (*Fig. 1*) into the main system. From point A the sewage is pumped to the disposal-works, situated on the Cape flats. The present population is 81,355, and the sewers have been designed to accommodate, with extensions, an ultimate population of 290,000, which figure will be reached about the year 1950 at the present rate of increase.

Adequate provision had already been made to carry storm-water directly to the streams; therefore the sewerage scheme is designed on

Fig. 1.

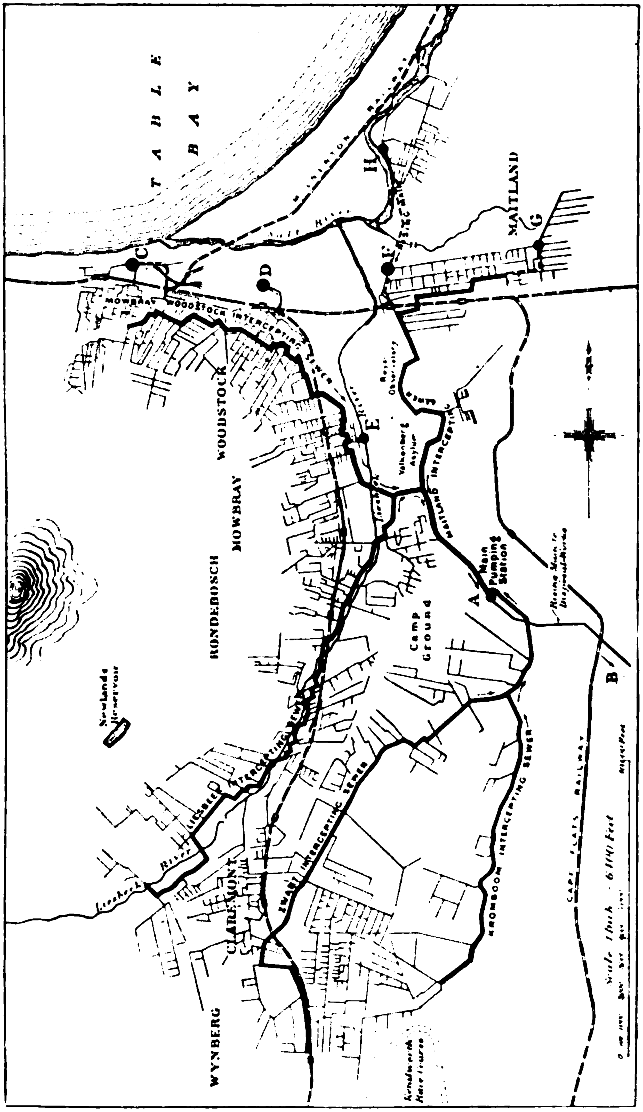


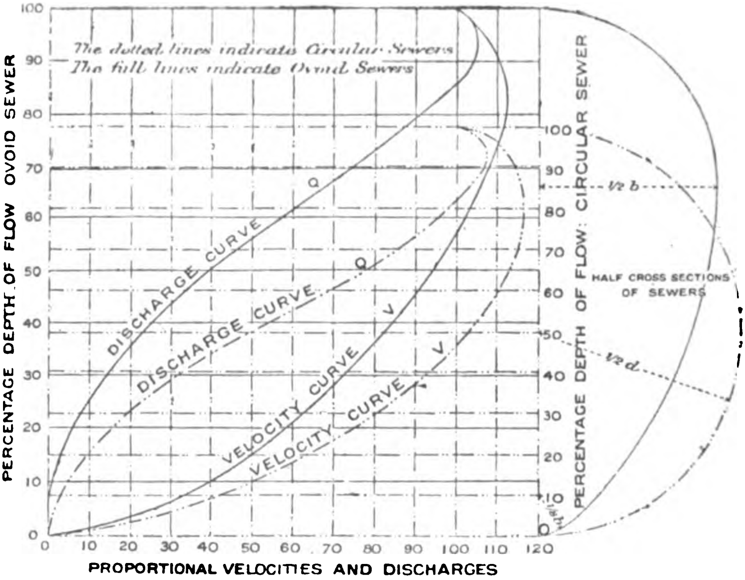
TABLE BAY
MAITLAND
WOODSTOCK
MOWBRAY
RONDEBOSCH
CLAREMONT
WYNBERG
Newlands Reservoir
Liesbeek River
Mowbray Woodstock Intercepting Sewer
Maitland Intercepting Sewer
Liesbeek Intercepting Sewer
Zwart Intercepting Sewer
Kromboom Intercepting Sewer
Royal Observatory
Valkenberg Asylum
Camp Ground
Main Pumping Station
Rising Main to Disposal Works
Cape Flats Railway
Kenilworth Race Course
Scale 1 Inch = 6,000 Feet
A
B
C
D
E
F
G
H

DRAINAGE SYSTEM.

the partial separate system, to discharge sewage proper plus rainfall from paved washing-areas limited to 100 square feet per house.

In a semi-tropical climate, with long periods of drought, it is of the utmost importance that the sewers should be self-cleansing with the minimum average daily dry-weather flow. The figure adopted for the design of limiting gradients is 25 gallons per head per day, which coincides with the minimum average consumption of water, and is 10 per cent. less than the average summer consumption after deducting water used for garden purposes. On the basis of this figure and the present population, the gradients have been arranged to secure, with the proportionate depth of flow, minimum velocities ranging from not less than 3 feet per second for the small pipe sewers to 2 feet per second for the larger culverts. In this connection the graphs shown in *Fig.* 2 may be referred to. The horizontal co-ordinates indicate the proportion of the total velocity or discharge of the full sewer to be expected at any depth of flow, the latter being shown vertically as a percentage of the depth when the sewer is full.

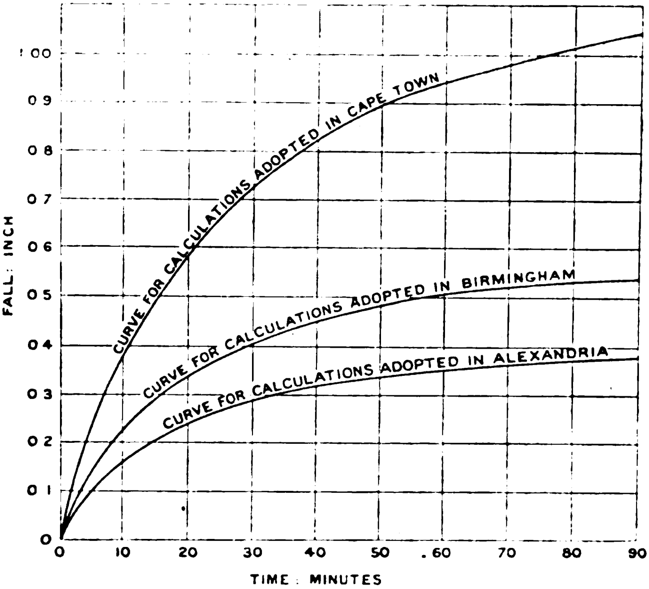
Fig. 2.



Maximum Capacity of the Sewers.—The elements upon which the maximum capacity of the sewers is based are the maximum sewage flow from the ultimate population during the life of the works, and the corresponding volume of storm-water to be admitted from the yards. The proportion of the maximum average summer consumption of water that will reach the sewers has been taken at 35 gallons per head per diem. The result of gaugings in sewers serving populations ranging from 500 to 70,000 in Birmingham, Alexandria (Egypt), and Capetown show that the highest average percentage flow in any 1 hour is 7·7. Therefore 8 per cent. in 1 hour, or half the total flow in 6 hours, will cover the maximum daily variations.

To the latter figure the storm-water flow must be added, and the next step is to ascertain the quantity of rainfall to be accommodated. Until recent years an arbitrary figure for rainfall has been generally used for the whole district covered by main-drainage schemes, with the result that the sewers have been incorrectly proportioned, the main trunks being too large and the branches too small, involving unnecessary expenditure. It is submitted that the more correct procedure is to graduate the sizes of the sewers to receive storm-water due to rainfall-intensities corresponding with the time of concentration of the discharge from the areas drained by them, the correct discharge for each sewer being calculated separately. For this purpose rainfall-intensity curves have been computed. *Figs. 3* and *4* give the intensity curves adopted for the scheme; they have been constructed from carefully-selected authentic records, and give results considerably higher than the

Fig. 3.



Birmingham and Alexandria curves. In *Fig. 5* the lower curve shows the rainfall-intensity discharges expressed in cubic feet per head, on the 100 square feet per house basis, and the upper curve gives the maximum sewage-discharge figure in combination with it.

Fig. 4.

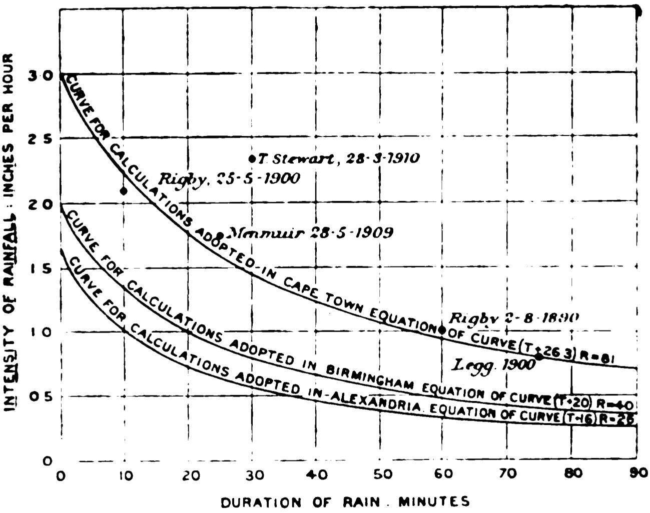
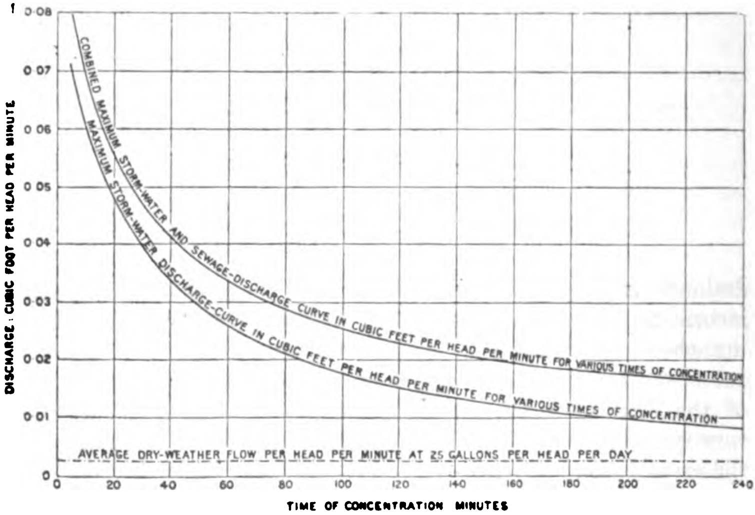


Fig. 5.



The combined curve has been employed for the maximum-capacity calculations, the population draining to, and the time of concentration of the discharge for, each particular sewer being the only other factors needed. The well-known formula by Messrs. Santo Crimp and Bruges was used for calculating the sewer-capacities.

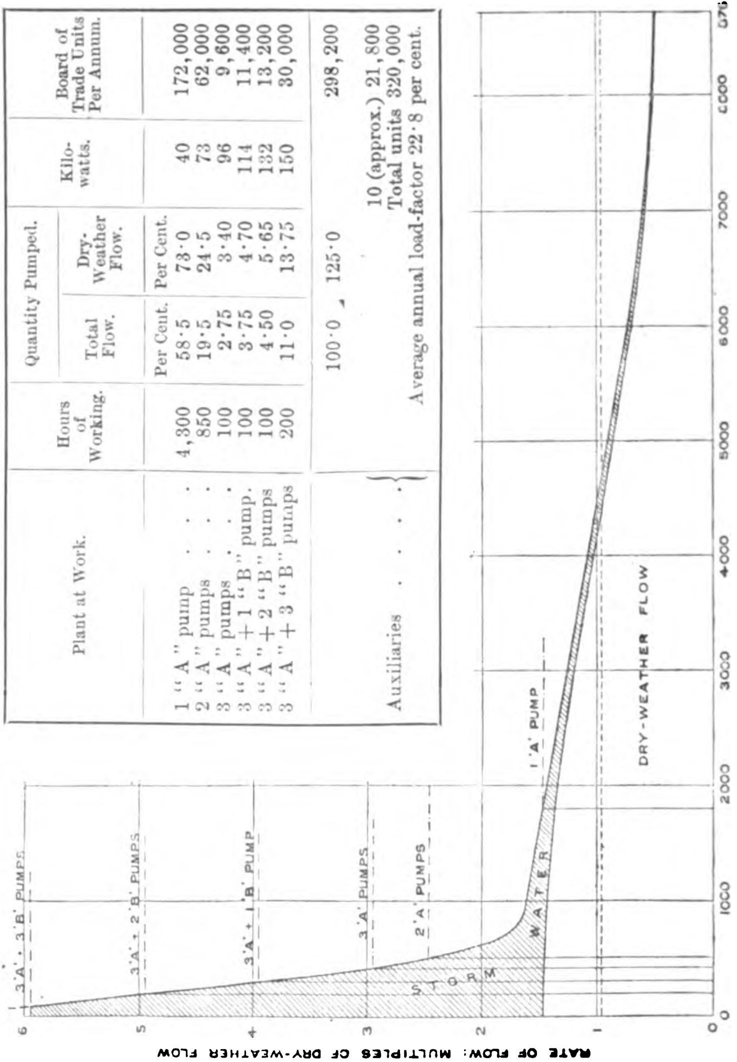
From gaugings taken by the Author some years ago in sewers in Birmingham, several of which had been constructed 30–60 years previously, it was found that the coefficient varied considerably, according to the condition and age of the interior. The result supported the inference that Mr. Santo Crimp's coefficient of 124 provided sufficient margin for reduced efficiency due to deterioration, in properly-constructed sewers.

Sewer Sections.—Up to 15 inches in diameter the sewers were laid with salt-glazed stoneware pipes of South African manufacture, and for diameters from 18 to 40 inches, including the ovoid sections, locally-manufactured granolithic concrete tubes were used, cased with ordinary concrete. Culverts above 40 inches in diameter were constructed in concrete in situ, the inverts being lined with either blue bricks or granolithic concrete. The ratio to the internal diameters adopted for the thickness of the larger sections are 15 per cent. at the crown, 25 per cent. at the springing, and 20 per cent. at the invert. The scheme comprises a total length of 120 miles of subsidiary sewers, 17 miles of intercepting-sewers, and 4 miles of rising mains. The house-drainage is being effected without the introduction of intercepting-traps, and up to the present time no complaints in regard to nuisance have been received.

Main Pumping-Station.—The position chosen for the main pumping-station at Raapenberg is centrally situated, and the pumping-plant is designed to deal with six times the dry-weather flow. The surplus storm-water which reaches the sewers in times of heavy rainfall is discharged through storm-water overflows into the various streams at convenient points. Before the sewage reaches the pumps it passes through detritus-pits which are emptied, when cut off, by a grab, the water being first drawn off to the pumping-chamber and discharged thence into the main pump-well. From the detritus-pits the sewage flows to the pump-well in the main station. The pumping-plant is housed in a central compartment surrounded by the pump-well, at a sufficiently low level to avoid the necessity for suction-pipes and priming. The inlets to the pumps are protected by electrically-driven revolving screens fixed inside the well. Three centrifugal-pumps, coupled to 50-HP. motors, deliver the sewage, up to three times the dry-weather flow, through a cast-iron rising main, to the disposal-works about a mile distant, against a maximum head of 52 feet. The surplus sewage, from three to six times the dry-weather flow, is lifted into the stand-by storm-water tanks, situated near the station, by three other pumps coupled to 25-HP. motors, against a maximum head of 27 feet. Two "Stereophagus" pumps are coupled to 25-HP. motors and are placed at the end of the station on a lower level than the rest of the plant, in order to

Fig. 6.

Total Annual Dry- and Wet-Weather Flows.

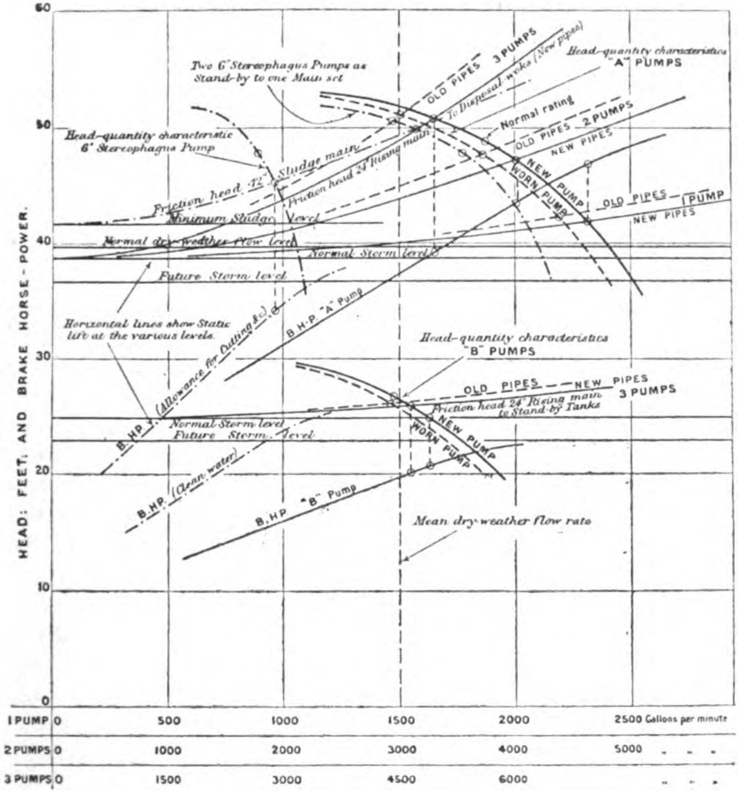


Plant at Work.	Hours of Working.	Quantity Pumped.		Kilo-watts.	Board of Trade Units Per Annum.
		Total Flow.	Dry-Weather Flow.		
		Per Cent.	Per Cent.		
1 "A" pump . . .	4,300	58·5	73·0	40	172,000
2 "A" pumps . . .	850	19·5	24·5	73	62,000
3 "A" pumps . . .	100	2·75	3·40	96	9,600
3 "A" + 1 "B" pump.	100	3·75	4·70	114	11,400
3 "A" + 2 "B" pumps	100	4·50	5·65	132	13,200
3 "A" + 3 "B" pumps	200	11·0	13·75	150	30,000
		100·0	125·0		298,200
Auxiliaries				10 (approx.)	21,800
				Total units	320,000
	Average annual load-factor 22·8 per cent.				

drain the floor of the pump-well and to raise the night-flow, together with the day's screenings, to the disposal-works. The pump-well is ventilated by electrically-driven fans during the summer. *Figs.* 6 and 7 give the head–quantity characteristics and the annual duty to be performed by the centrifugal pumping-plant. The average annual load-factor is 22·8 per cent.

Stand-by Storm-Water Tanks.—The reinforced-concrete storm-water tanks are 79 feet in diameter, with a mean depth of 5 feet, and their combined capacity is equal to one-quarter of the average daily dry-weather flow. The diluted sewage is admitted through a bell-mouth shaped inlet near the centre and flows radially towards the outer channel which skirts the perimeter at water-level, whence it is discharged into the stream nearby.

Fig. 7.

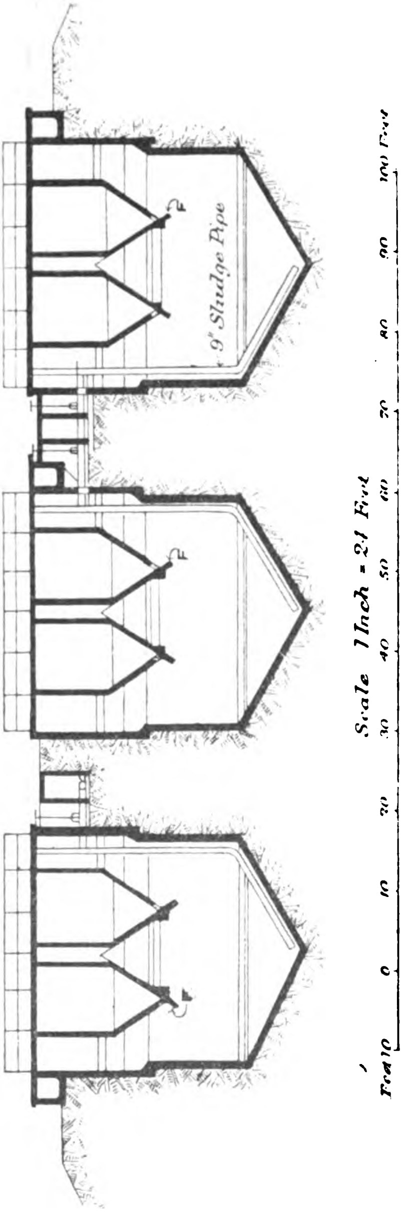


HEAD-DISCHARGE CHARACTERISTICS: MAIN PUMPING-STATION.

After the rainfall ceases the top water is decanted through a floating arm, and the sludge and silt are returned through the sludge-pipe to the pump-well at the main station, where they mix with the sewage proper and are pumped to the disposal-works. Dwarf guide-walls are constructed on the floors of the tanks to facilitate cleansing.

Sewage - Disposal Works.—The site of the disposal-works, situated to the west of the suburbs and on the far side of the Cape Flats Railway, was selected owing to the suitability of the land for irrigation, to its comparative isolation, and to the direction of the prevailing summer winds, which blow from the south-east quarter and frequently with considerable violence. The small stream, towards which the farm drains, is practically dry for more than half the year, rendering it necessary to take precautions to produce a stable effluent at the works. The system embraces sedimentation and

Fig. 8.



CROSS SECTION
SEDIMENTATION-TANKS.

sludge-digestion tanks, percolating bacteria-beds, humus-tanks and land irrigation, and is designed to avoid septicization of the tank effluent as far as possible, in order to secure the best results from the bacteria-beds and to obviate nuisance during the process of purification.

The reinforced-concrete double-storey tanks (*Fig. 8*) are designed to deal with the whole of the daily dry-weather flow in 18 hours. The combined contents of the upper chambers are equal to 8·33 per cent. of the volume of the daily flow, and the average velocity through them is about 0·64 foot per minute. The lower twin chambers are large enough to contain 6 months' supply of sludge, which is decanted, after digestion, and conveyed by gravitation through a reinforced-concrete main to the pump-well, situated beyond the humus-tanks. Spray pipes are fixed in the course to facilitate sludging. The total depth of each tank is 34 feet and the diameters of the twin chambers are 28 feet. Provision is made for reversing the flow through the upper storeys as required. A special feature of the upper storeys is a lip (*Fig. 8*) projecting beyond the slots, to direct the gas towards the vent chambers and to prevent subsidence from being disturbed. Very bad ground and a considerable volume of water were encountered in the lower excavations during construction, and a preparatory floor had to be laid with concrete slabs before the reinforced concrete forming the inverted cones could be placed.

Percolating Bacteria-Beds.—These beds have a combined area of 2¾ acres. The concrete floors of each bed slope towards a central culvert traversing its length, which provides access for inspection of the effluent issuing from any section of the bed. The false floors, on which the medium is laid, are constructed with perforated concrete slabs resting at the corners on concrete cubes. The sewage is sprayed evenly over the medium by travelling distributors actuated by water-wheels fed by siphons from a central trough. This method appears to be more suitable for a warm climate than are fixed jets, which are liable to cause nuisance under unfavourable conditions.

When the system was first designed it was intended to adopt broken granite for the medium, but the remarkable results obtained with brushwood at Toronto, through which tank liquor has been successfully treated at a much higher rate than through stone, induced the Author to substitute the latter for the central bed. Large quantities of Australian wattle, which appeared to be suitable for the purpose, grow on the Cape Flats in the immediate vicinity of the works. This was cut, pressed into bales, and placed in the bed to a depth of 5 feet 6 inches, the bales being so arranged as to break joint. Up to the present the result with this medium has been fairly satisfactory, but it has not approached the degree of efficiency attained with the Toronto bed. This is probably due to too rapid percolation, which is being corrected by placing much finer bundles on the surface as the brushwood subsides.

Humus-Tanks.—A design similar to the primary tanks has been adopted for the humus-tanks, but on a smaller scale. The humus is decanted to the pump-well, and, after being mixed with the sludge from the sedimentation-tanks, is pumped with the latter to the drying-beds. It is hoped that the difficulty in de-watering the humus, due to its colloidal nature, will be overcome by mixing, but should it eventually prove to be unsuccessful, owing to local conditions, the two sludges can be dealt with separately by the same plant.

The farm, 550 acres in extent, is irrigated by the effluent from the humus-tanks. Owing to the sandy nature of the subsoil the land absorbs water freely and is very fertile. Screens consisting of belts of trees have been preserved around the disposal-works and irrigation-paddocks.

The cost of the present scheme, when it is entirely completed, will amount to about £810,000.

Mr. J. Duncan Watson, M. Inst. C.E., Chief Engineer to the Birmingham, Tame and Rea District Drainage-Board, watched the Council's interests in connection with the plant purchased overseas, and the Author wishes to acknowledge his assistance with the detail designs for the pumping arrangements, and to thank him for his opinion upon the disposal-works. The thanks of the Author are also due to Mr. W. N. Hoskins, Assoc. M. Inst. C.E., Assistant Engineer in charge of the scheme, and to Mr. Angelini, Chief Draughtsman, for the preparation of the drawings to illustrate the Paper.

The Paper is accompanied by seven sheets of tracings and three prints, from some of which the Figures in the text have been prepared.

Discussion.

The PRESIDENT moved a vote of thanks to the Authors. The President.

Mr. TICKELL remarked that he would like to point out the economic value of drainage-schemes, because he had never seen a published reference to that important subject. Since writing his Paper he had come across some statistics which threw considerable light on the matter. Money spent upon a drainage-scheme was always put down as unremunerative expenditure. An eminent engineer had said to him recently that in dealing with water-schemes one was dealing with a saleable commodity, but a drainage-scheme was merely getting rid of a waste product. That was the commonly-accepted opinion, and consulting engineers knew that it carried weight with authorities who dealt with the allocation of public funds. He thought it was an entirely mistaken point of view, and he believed the study of health-statistics would prove that drainage-schemes might be more remunerative than most of the public works which were said to be so. The economic value of the drainage was the benefit drived from a reduction in the rate of sickness. Sickness was a dead loss to the community, and, in the case of a bread-winner, a fair valuation of that loss would be, at the very least, his keep, or the amount of his earnings during the period of sickness. In England the loss was met, in some cases, by benefit societies. In the East it was paid for either by the hospitals or by a man's relatives. The statistics referred to showed a relationship between the death-rate and the rate of sickness, three examples of which were given in the following Table, which dealt with an aggregate of 5,174,960 years of life observed. Mr. Tickell.

	Sickness.	Deaths.	Ratio: Time of Sickness equivalent to One Death.
	Weeks.		Weeks.
Manchester Oddfellows	7,022,475	39,061	180
Hearts of Oak	1,452,106	7,853	185
Leipzic	1,545,613	8,668	178
Total	10,020,194	55,582	180

Mr. Tickell. Those figures had been dealt with to some extent in medical papers and were to be found in the Journal of the Royal Statistical Society.[1] It was therefore fairly established that 180 weeks of sickness ocurred for every death. The population of Colombo was 300,000. The drainage-scheme had made a reduction of six per thousand in the death-rate, which meant a saving of 1,800 lives per annum owing to the improvement in the health of the community. Assuming that funerals cost, on the average, about Rs.100, the saving to the community in funerals alone was Rs.180,000. Assuming 180 weeks of sickness per death, 324,000 weeks of sickness were saved per annum. To estimate the saving of money from that cause, he assumed the average keep per person at Rs.5 per week. The actual expense to the community was, of course, much greater, on account of doctors' fees and medicines. Thus the saving on the reduction of sickness which could be fairly attributed to the drainage-scheme amounted to Rs.1,620,000 a year. The total saving came to Rs.1,800,000, and that represented 10 per cent. on the outlay upon the drainage-works. He thought that figure would stand criticism. When he first went to Colombo he found that the municipality, instead of having a debt, had actually Rs.1,000,000 surplus in hand. He pointed out to them that it was an anomalous state of affairs, as they were doing nothing for posterity; and before he had been there many years the municipality were reported to be in a state of insolvency! Since he had come away the municipality had recovered their financial stability and now had funds in hand to build a very fine town-hall. Although the financial recovery might be attributed to his departure from the island, he thought the figures he had given went to show that a truer explanation was the benefit resulting from improved sanitation.

Professor Snape. Professor A. E. SNAPE remarked that, as he had had experience in sewerage and sewage-disposal works both in Great Britain and in South Africa, he was interested in the two Papers on South African practice. Sewerage design in South Africa was very similar to British practice, except that the intensity of the rainfall was greater. It had to be remembered that the rainfall of 30 inches a year mentioned by Mr. Jameson fell mainly in about 4 months, and that 8 months of the year were practically dry. Heavy discharges meant very large sewers, which would be working at their full capacity at infrequent intervals. It was true that the separate system was universally adopted in South Africa, but the rainfall

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[1] E. A. Rusher, "The Statistics of Industrial Morbidity in Great Britain." Vol. lxxxv (1922), p. 27.

Professor Snape.

on polluted areas had to be taken into the sewers, and this meant that balance and judgment had to be displayed in settling the design. But the main difference between British and South African practice was in the matter of sewage-disposal. South Africa had a hot, semi-tropical climate, and there were many months of dry weather, when water was a most valuable commodity. The use of water in South Africa was subject to many legal restrictions. Many of the rivers were absolutely dry for months at a time, or their flow was very small. Under the law, the use of water from a public stream was divided into three categories. The primary use of water was for domestic purposes and for the watering of stock; the secondary use was for irrigation; and the tertiary use for industrial purposes, water-power, etc. Before using any water from a public stream for irrigation, all the requirements of the riparian owners along the stream had to be satisfied for the primary use of water; and before using it for tertiary purposes, all the primary and secondary requirements of water had to be satisfied. The domestic purposes could be satisfied fairly easily, but the irrigation requirements were more difficult to meet. It was being felt now that, with the industrial development of South Africa, the rule that before water could be used for industrial purposes the riparian owners had to be satisfied with regard to irrigation was a very difficult matter. He mentioned that point because there had been several law cases with regard to turning an effluent into a stream where every riparian owner had the use of the water for domestic purposes. Lawyers had laid down the dictum that that water was polluted unless it was fit for drinking when turned into the stream. That meant that engineers responsible for sewage-disposal works had had very great difficulties to contend with, because, even when conforming with the very high standards of the British Royal Commission on Sewage-Disposal, the effluent would not be accepted as drinking-water; therefore the argument was that it was polluted, and damages and an injunction could be claimed against the municipality. Fortunately, the difficulty could be, and was, solved in most cases in South Africa by having no effluent at all, and that was the condition that had to be aimed at in order to keep absolutely clear of the legal difficulties. Both Johannesburg and Bloemfontein had no visible effluent. The water was turned on to the land, the land was cultivated, and a buffer area was provided around the cultivated area, on which trees were generally grown. If there was no visible effluent, pollution could not be proved. The Johannesburg sewage-disposal works at Klip Spruit were about 1½ mile from the Zwartkopjes pumping-station of the Rand Water Board, and very regular

Professor Snape. analyses of the water were carried out. Although the sewage-works were several hundred feet higher than the pumping-station, no pollution of the water had been proved. He did not think there was any doubt that the water was quite pure. Such conditions rather simplified the problem of sewage-disposal in South Africa. A large area was usually in the possession of the municipality, having been given by the Government when the town was laid out. Water was so valuable in South Africa that it should not be wasted, but utilized for the growth of crops as far as possible. Crude sewage, however, could not be put on land. "Sewage sickness" and a sort of "macintosh" effect over the land had occurred in South Africa as well as in Great Britain. Therefore it was necessary to provide very careful treatment to eliminate as far as possible all the suspended matter, and, on certain lands, dependent on the texture of the soil, to transform the colloidal matter in the sewage. For most of the works a tank of adequate capacity and treatment on the land were sufficient, and bacteria-beds were not generally necessary because the large area of land under cultivation performed the same operation. The Pretoria works were a peculiar case, because, owing to many difficulties, it was not possible to obtain a large quantity of land; and Mr. Jameson had had to lay down a rather elaborate series of works to produce an effluent of very high standard. It might be asked why the Pretoria municipality had been free from litigation. It so happened that the riparian owners for 5 miles or so below the outfall were all market-gardeners, and the land was intensively cultivated by them. They found that the effluent from the sewage-works was not only very valuable as a fertilizer, but in the dry season it gave them additional water. He presumed that beyond that limit the riparian owners obtained water purified as a result of its use for irrigation. In Capetown the effluent ran into a stream only a mile or two from the sea, and no river water was required for domestic purposes. Under the altered conditions in South Africa the engineers were not slavishly copying British details. Principles had to be studied and applied to those altered conditions. As a rule, the sewage in South African towns was highly concentrated in comparison with sewages in Great Britain. For example, the Johannesburg sewage was four times as strong as the Birmingham sewage, both because the stercus-pails from unsewered areas were tipped into the sewers, and because rain-water, during the many months of the dry season, did not dilute the sewage. A concentrated sewage had to be dealt with by different means from those common in this country. Some of the early works put down in South Africa, following British practice, were not a success, for those reasons. He noticed that the

Professor Snape.

tank-treatment at Pretoria differed from that adopted at Capetown. Sedimentation-tanks were used in both instances, but Mr. Jameson discharged the sludge twice daily from the sedimentation-tank into a separate tank, which he called a septic tank, for sludge-digestion; whereas Mr. Lloyd-Davies had put down double-storied tanks, and allowed the sludge to remain in the lower chamber for 6 months. The Capetown works were not yet working at their full capacity. The works were finished, but not half the houses were joined up, and it would be 3 or 4 years before all house-connections were finished. He favoured the ejection of sludge from the sedimentation-tank as fresh as possible. That seemed more cleanly. He would be pleased to know why Mr. Lloyd-Davies allowed the sludge to remain in the lower chamber for 6 months. He congratulated the Authors upon valuable contributions, which gave, almost for the first time, a description of modern sewage-disposal practice in South Africa.

Mr. O'Shaughnessy.

Mr. F. R. O'Shaughnessy thanked The Institution for giving him an opportunity to take part in the discussion of such interesting Papers. He congratulated Mr. Tickell on bringing forward an exceedingly cogent argument for the business men who controlled expenditure in connection with drainage-works, because the thoughtless objection was frequently raised that such works showed no return compared with such public services as water-supplies. The Papers presented different conditions in various parts of the Empire, one of the places referred to being in tropical country, the other practically sub-tropical, and the third in a more or less temperate climate. The Emscher-Imhoff, double-storied tank, first installed at Essen, became very popular in America for a time, but there was not much information available with regard to its working under different conditions. In 1913 he visited Essen, and Dr. Imhoff showed him round a number of works there. The one defect which struck him at the time was the question of dealing with storm sludge. In the Emscher-Imhoff tank, instead of putting the sludge concentrated from the flowing sewage into the digesting tank in large doses at once, it was put in very gradually, and was taken up by the tank as it settled from the flowing sewage in the upper chamber; the storage below had therefore to be so large as to allow of complete digestion of the sludge, in order that its offensiveness might be destroyed and especially its colloidal character changed. The offensiveness might be dissipated by digesting the sludge in a comparatively short time, but the changing of the colloidal character was an exceedingly slow process. That physical change was necessary in order that when the sludge was put out on the drying-area it should dry in a reasonable time. Dr. Imhoff claimed that

Mr. O'Shaughnessy. his original tank could produce absolutely inodorous sludge, quite black, and exceedingly mobile because it contained a quantity of occluded gas, and that it could be dried in a matter of days and removed with the spade. It would be informing to know the behaviour of the Imhoff tank under different conditions. Interesting information was given in Mr. Tickell's Paper with regard to mishaps. He took it that the pressure of the gas in one of the tanks had become so great that it practically disrupted one of the partition-walls. Mr. Jameson, from his experience of the Imhoff tank, condemned it, and was quite sure it would not be generally adopted throughout South Africa. On the other hand, Mr. Lloyd-Davies had actually installed such tanks at Capetown; they were just beginning to come into operation, and it would be interesting to hear later on whether his experience confirmed Mr. Jameson's emphatic opinion. Mr. Lloyd-Davies proposed rather novel procedure, about which Mr. O'Shaughnessy was somewhat sceptical. The humus from the filters was of a highly colloidal character and much more retentive of water than was ordinary sludge. Mr. Lloyd-Davies did not give details, but he proposed to mix that highly colloidal humus with highly digested and presumably non-colloidal sludge from the Imhoff tank. It did not seem consistent procedure, after going to all the expense and trouble of providing tanks to render the sludge non-colloidal, to mix with it probably 20 to 25 per cent. of exceedingly colloidal matter. Mr. Lloyd-Davies referred to a bacteria-bed filled with wattle, and Mr. O'Shaughnessy believed that something of the nature of wattle had been used at Toronto. Long before the percolation filter was adopted for treating sewage, vinegar manufacturers were familiar with the use of such material in connection with the oxidation of alcoholic mash. They distributed the alcoholic mixture by sparges over bundles of twigs, and the organisms which produced the change grew on the twigs. It was interesting to see that that principle was being tried in South Africa. If such a medium were convenient and cheap, there was no reason why it should not prove very successful. Mr. Jameson's reference to the work done at Birmingham by Mr. Watson and himself had not, he thought, made their practice perfectly clear. They prevented septic action as far as possible in the whole of the sewage and the sewage-liquor in course of treatment, before it was put on to the oxidizing bacteria-beds. Every possible endeavour was made to eliminate septic action, because it meant the fouling of the liquid and an increase of nuisance when it was exposed to the air. After being passed through a grit-chamber, the sewage went through sedimentation-tanks, primary and secondary, and flowed

Mr. O'Shaughnessy.

to bacteria-beds 5 miles distant, in a large sewer. As a further screen to the bacteria-beds, deep, inverted conical tanks were installed for a tertiary sedimentation of the sewage liquor. It had been found that they were not only useful but absolutely essential if successful working of the beds was to be achieved. The tanks were installed nearly 20 years ago under Mr. Watson's supervision; they had been a source of great interest, and it was gratifying to see that Mr. Jameson had found them so satisfactory in practice. They were capable of yielding very good results, but they needed to be worked intelligently and with very great care—much more care than an ordinary rectangular sedimentation-tank. The theory of the inverted cone or pyramid form of sedimentation-tank had never, as far as he knew, been adequately dealt with. He had found it a very baffling problem, and it required the expert attention of those skilled in mathematics and hydraulics to work it out on scientific lines; but their attention did not seem to have been drawn to it, or they had not been sufficiently interested in it to take the matter up. It had a very important practical bearing, because there was great diversity of opinion among engineers as to how the flow of liquid should be arranged in an upward-flow tank. Mr. Jameson referred to the fact that Mr. Watson had adopted upward-flow tanks (they were called Dortmund tanks until a deputation from Dortmund visited his works and disowned them; and he was sure Mr. Watson would be quite content to accept the credit which Mr. Jameson gave him: certainly Mr. Watson had taken a great deal of pains and care to perfect the working of those tanks). The drawback with them, as Mr. Jameson pointed out, was that, with crude sewage of a rather dense character, the sludge was liable to become consolidated in the sump and to present difficulty in working. Mr. Watson and he had found, in common with Mr. Jameson, that such a tank was peculiarly liable in the summer time to set up fermentation. The rather large surface in relation to the volume of the tank favoured the formation of an exceedingly active bacterial layer, which caused great trouble by making the liquid foul. If the tank were emptied, a thin black layer would be found on the walls; it was probably no thicker than if put on with a whitewash brush, and yet it was quite sufficient to foul the whole of the water. Mr. Jameson, who treated crude sewage in those tanks, had found that if they were not sludged frequently the sewage became septic, the tank reversed its action, and the sludge rose in a mass to the top, passed forward to the bacteria-bed, and caused considerable damage. From the interesting remarks of the previous speaker and the figures in Mr. Jameson's Paper, it was quite evident that the

Mr. O'Shaughnessy. conditions in South Africa were not comparable with the conditions in England. Nevertheless, he had gone into the question how the average sewage compared with the sewage at home. The Pretoria sewage was certainly on the strong side; it was rather more highly ammoniacal, probably because more septic action had gone on in the stercus; but apart from that it was not really stronger than what might be called an average sewage in this country. There were one or two curious things in Mr. Jameson's figures. The average of the suspended matter was about 40 parts per 100,000, and the average effluent from the Watson tank contained as much as 14 or 15 parts per 100,000 of suspended solids going forward to the bacteria-beds. It was not commonly appreciated that that would mean, if a reasonable allowance were made for the other sludge-forming material (colloids), that the additional matter which would come down on the bacteria-beds as sludge was about 10 parts per 100,000; so that over 20 parts per 100,000, out of a total of 50 would be put on the bacteria-beds. Yet, in spite of that, Mr. Jameson said that he did not need a humus-tank, and only needed to flush his bacteria-beds about once a year. When a bed went out of action, as the result of being choked with solids and colloidal matter, he allowed it to stand. To use a bacteria-bed for sludge-disposal in that way was an extravagant procedure, and more information was required than appeared in the Paper to enable the cost to be compared. Whilst criticizing Mr. Jameson's work, he liked his method of tackling such difficult questions, particularly his originality in tackling the sludge question. Every engineer and chemist who had to deal with sewage knew what a bugbear the sludge question was. Mr. Jameson's method of attack seemed to be purely empirical, but he had obtained on the whole successful results. From his own knowledge of sludge-disposal and digestion he would say that possibly in Mr. Jameson's tank there was a fairly large quantity of paper and other cellulose matter and gas-forming material, which caused a very substantial portion, or practically the whole of it, to come up with the scum; it was lifted out of the region of the actual digestion going on, and in the high temperature of Pretoria the scum dried rapidly. Thus it developed into a condition in which it could be dried for a manure. It was necessary to be quite clear, with regard to sludge-disposal, whether the aim was to produce a manure, or merely to get rid of a very troublesome and obnoxious substance. Apparently Mr. Jameson had struck a mean course and obtained a sludge which had a very high nitrogen-content, showing that the degree of digestion which it had undergone was comparatively small. With regard to the nuisance of crawling flies, that was a sure sign of improperly digested sludge, because a sludge undergoing vigorous anærobic fermentation would be found incapable of harbouring a single fly. Flies in bacteria-beds were bad enough, but they were very dangerous in connection with sludge, especially in a climate such as that of Pretoria. **Mr. O'Shaughnessy.**

Mr. G. MIDGLEY TAYLOR remarked that Mr. Jameson's Paper apparently dealt with what was more or a less a stereotyped system of sewage-disposal 20 years ago. It contained no reference to the modern researches of Drs. A. M. Fowler and E. Ardern in Manchester, Mr. Howarth of the Sheffield sewage-works, and others, or to the work in connection with activated and agitated sludge which had been done in America. He regretted Mr. Jameson's use of the name Watson in connection with what was known universally as the Dortmund tank. Mr. Jameson said that he was definitely of opinion that the Emscher-Imhoff and Travis designs would not become standard South African practice. As a commentary upon that, Mr. Taylor drew attention to the fact that Mr. Tickell, after having struggled with a variety of tanks, came to the conclusion that the Emscher-Imhoff was the only one suitable for his climate, and Mr. Lloyd-Davies had also adopted that type. In connection with the Aapies stream, it was stated that the daily flow of sewage was 1½ million gallons; the original dry-weather flow of the Aapies stream had been 1½ million gallons, and it was now 4½ million gallons. Perhaps Mr. Jameson would explain that rather cryptic statement. The effluent obtained from the Dortmund tank contained little more than half of the suspended solids that went in. An ordinary Dortmund tank that did not keep back more than half the solids that went into it was not functioning properly. A little septic action was taking place, because the albuminoid nitrogen-content of the inflowing sewage was 1·00 part for 100,000, while that of the tank effluent was 0·88. The saline ammonia was 3·00 as it went in and 3·25 as it came out. The nitrogen-content was practically identical. The works were divided into four compartments, which meant that there were four screen-chambers, four grit-chambers, etc. He asked whether it would not have been much cheaper to unite the different sections of the works, using single units, or a single series of units, which would simplify and cheapen the management. He was afraid Mr. Jameson rather libelled English practice in stating that his obsolete type of grit-chamber (*Fig. 3*) had been adopted therefrom. Mr. Midgley Taylor had never seen such a grit-chamber. He considered that the removal of the dry-weather grit-content was unnecessary. He had yet to learn that ordinary domestic sewage had any appreciable quantity of grit in it, and he suggested **Mr. Taylor.**

Mr. Taylor. that the whole of the contents of the sewers, after passing through the screen, should be led directly into the Dortmund tank. The suspended solids in the effluent from the oxidizing-beds were given as 2·6 parts per 100,000, in spite of the fact that 14·2 parts per 100,000 were entering the beds. It appeared that the beds were retaining an undue quantity of suspended solids, or that something was wrong with the analysis. The albuminoid ammonia-content of the final effluent was about twice as great as was generally adopted by the Rivers Board standards throughout Great Britain. He did not allude to the Thames or Lea Conservancies, which had standards of their own, owing to the use of those rivers for drinking-water. The dissolved oxygen absorbed in 5 days at 65° F. was 2·20, which was 10 per cent. below the standard laid down on p. 87. While the temperature at which the sewage reached the works was unknown, very definite conclusions could not be drawn; but he gathered that the temperature must be fairly high, and the results, considering that only 75 gallons per cubic yard per day were passing the oxidizing-beds, were far from satisfactory. Possibly that might be accounted for by a remark to the effect that, when the inflow of the sewage on to the beds was stopped, the beds drained themselves, and air filled the voids temporarily freed from liquid. No portion of a properly-constructed bed should ever be full of liquid, which should trickle down through the spaces of the material. He asked why Mr. Jameson did not use a humus-tank instead of hosing out the beds annually. He would be interested to know the quantity hosed out in that time. Since the Pretoria works were in a climate in which 8 months of the year were absolutely dry—as regarded both rainfall and atmosphere—and since 10 acres of land were available, it seemed to him that sludge-disposal could have been carried out very much more quickly by spreading the sludge over small sludge-areas in thin layers, using the sludge-tanks only in the wet weather. Mr. Tickell's Paper was an interesting description of big works carried out under adverse concumstances. He had learned from Mr. Tickell that the temperature at which the sewage reached the works was something like 85° F., which was what might be expected in a very hot climate. He wished to know whether any analyses had been made. The Table in *Fig. 6* of Mr. Lloyd-Davies's Paper was very interesting. From that it would be noted that, although the intensity of rainfall in Capetown was occasionally very much higher, as was disclosed by *Fig. 4*, than that which obtained in England, yet, owing to the 8-month dry-weather period, during the whole year it was necessary to pump only 25 per cent. more than the average dry-weather flow of sewage, although six times the dry-weather flow was dealt with at times. That was useful information, because in England, where such intense rainfalls did not occur, but where the rainfall was spread over long periods, engineers had reckoned 50 per cent. more than the average dry-weather flow. Mr. Lloyd-Davies called attention to certain projections in his tanks. Those were not new, but no doubt Mr. Lloyd-Davies had found them useful from experience. Mr. Lloyd-Davies had chosen an area of 550 acres of land of a sandy nature, which absorbed water freely and was very fertile. Had it been found absolutely necessary to remove the solids in the double-series tank; or would it have been possible, without the interposition of tanks, to deal with the sewage in its crude condition, with great advantage to the profits which might have been obtained on the agricultural products? **Mr. Taylor.**

Mr. Cotterell. Mr. A. P. I. COTTERELL remarked that the Papers were very welcome, as it seemed to him to be some years since sewage-treatment was last discussed at the Institution. Sewage was such a variable article that uniformity of treatment would never be obtained, and there would be always advocates of different methods. Mr. Jameson emphasized the importance of the primary tank. He agreed with Mr. Midgley Taylor in asking, with all due respect to Mr. Watson, that the name Watson tank should be dropped. Mr. Jameson's tank was the old vertical or Dortmund tank that had been used for primary treatment for many years, long before the Watson tank was built. He was sure that Mr. Watson would be the first to disclaim authorship of vertical-settlement treatment in primary tanks. He himself had used vertical-flow tanks 30 years ago, and 25 years ago he had tried very much the same system as described by Mr. Jameson. He had long considered that the vertical flow of sewage carried out the processes in one-fourth of the time required by a horizontal-flow tank. It seemed to him, from the figures given as to the population upon the sewered area and the daily dry-weather flow, that the sewage was moderately diluted. There was apparently a good deal of water at about 2 a.m., which Mr. Jameson said was almost pure water from flushing-cisterns. He did not think it would have been possible to deal with the night-soil deposit in the manner described had not there been a very large amount of dilution in the sewers, which increased the sewage-flow to 50 gallons per head per diem. The remarks with regard to the mixture of foul septic liquor and sweet sewage were quite to the point. Mr. Jameson said that, unfortunately, the lesson of not mixing these together had not been fully learned in South Africa. Nor, for that matter, had it been entirely learned in England. The first idea in the development of the vertical-flow primary tank might be ascribed to Germany,

Mr. Cotterell. which produced the Dortmund tank, but England was not long in adopting similar methods. The ordinary Dortmund shape with the inverted cone bottom was used first, and its disadvantages were soon observed. Dr. Travis, in the Hampton tank, introduced a second chamber, whereby he was able to separate, to some extent, the septic liquor from the fresh sewage coming into the tank; he made the error, however, of allowing both the liquid from the septic or lower storey as well as that from the upper storey to flow away to the filters and so to spoil the effluent that he might otherwise have obtained from his first tank. Dr. Imhoff discovered that fault and separated the septic liquor in the lower storey from the fresh sewage in the upper portion, and thus made a distinct advance, of which the Americans were not long in taking advantage. That stage seemed to have been reached in England, but not much further advance had been made, probably owing to the war. It was now necessary to face the question whether a double-storied tank was the type that was fit to survive in the primary treatment of sewage. He was pleased to see that Mr. Lloyd-Davies had dealt with the intensity of the rainfall. Engineers owed him a debt for his investigations at Birmingham and Alexandria on the intensity of rainfall and discharge-formulas applicable thereto. Mr. Lloyd-Davies made the interesting announcement that he had adopted for filter-beds Australian wattle, or a sort of plaited withy—a method somewhat similar to that which had brought remarkable results at Toronto. That was no doubt a splendid way of building up an oxidizing-filter. The idea had been made use of many years ago by the late Mr. F. W. Stoddart in his drop-by-drop filter. He believed Mr. Stoddart had copied it from the methods used in making vinegar. An interesting Note[1] had been read at the last Engineering Conference of The Institution (1921), describing the treatment of straw with the effluent from primary tanks, by which a manure was obtained which was described as at least equal to farmyard manure. Withies would not make manure, but the principle was the same, namely, the introduction of as much air as possible to the percolating sewage, so as to oxidize it more completely than could be done by a granite or clinker-filled filter. The methods of sludge-treatment described were the most interesting part of the Paper by Mr. Jameson, who was very insistent on the immediate removal of the solids from the primary tank into septic tanks. Mr. Jameson said that, so far as he could judge, sludge removed under the double-storied methods in temperate climates persistently resisted the extraction of moisture under a natural atmospheric drying-process. He would like to know more about that, because it seemed to him a question which engineers might be asked to investigate further. Mr. Watson was making use of gas from a septic tank to drive a gas-engine, and perhaps a double-storied tank and the method of obtaining gas in that way might have a distinct advantage, by utilizing a gas which, in the open septic tank or on the drying-bed used in England, was simply dissipated into the atmosphere. It had been found experimentally that 10,000 to 20,000 cubic feet of gas could be obtained from a million gallons of sewage; whilst at Paramatta, Australia, 3 cubic feet of gas and more per head per day was stated to have been generated. This was a distinctly valuable asset. If the double-storied tank, or the tank in which decomposing sludge was confined, enabled that gas to be obtained, it was one point in favour of a confined tank. Mr. Jameson's account of the steps taken by which he managed finally to get an inoffensive sludge had its counterpart in England. At various works improvements had been introduced, the leading object being to reduce the water-content, and thus to render the sludge both manageable and inodorous. Air drying had done it, the moisture being thus reduced from 90 per cent., and even more, to 70 per cent. and less. Only that day, at Surbiton, he had been looking at some sludge-beds formed of clinker, under-drained, on which the sludge was pumped to a depth of 9 inches. That system was very satisfactory. Sludge could be removed within a month in the winter, and within a week in the summer; it was absolutely inodorous, had a high fertilizing-value, and could be sold at a good figure. He would have liked to hear more about the difficulties which Mr. Tickell had encountered in connection with the extremely bad subsoil in Colombo. He was pleased to see Mr. Tickell claiming as an asset, against the large outlay made on sewerage, the saving that had been effected in funeral-expenses. He had no doubt that it was a substantial item, curious though it might appear. It was difficult to see revenue in sewage-treatment works, although such works were as important as waterworks; but there were important savings, and it was a good thing to hear an engineer laying claim to them as a result of his work. Mr. Cotterell.

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[1] E. H. Richards and M. G. Weekes, "Straw Filters for Sewage-Purification." Proceedings of Sect. VI, p. 21.

Mr. G. W. Humphreys remarked that both Mr. Jameson and Mr. Lloyd-Davies said that in the systems they had installed they had done away with intercepting-traps, with good results in each case. Mr. Tickell did not mention what had been done in that respect in Colombo. There had been a movement to get rid of the intercepting-trap, but it was a matter on which he personally had Mr Humphreys.

Mr. Humphreys. no very definite opinion. So far as England was concerned, its existence, he thought, helped to keep out the common rat; he did not think a rat would negotiate an intercepting-trap in the dark, although it would when it could see light on the other side. Rats were fairly universal in their habitat, and he supposed that neither Capetown nor Pretoria was free from them. There could be no two opinions on the benefit derived from increased ventilation: the more ventilation that could be put into a sewerage-system the better it would be. The Pretoria Paper referred to an inland town, and Professor Snape had described very vividly the condition of affairs at Pretoria—how water was such a valuable commodity that the desideratum was to have a sewage-works with no effluent to foul existing streams, and that this result had been obtained in some South African towns by absorption; possibly this was the explanation of the figures Mr. Midgley Taylor had called attention to, with regard to the flow in the Aapies river—that the river had more water in it since the sewage-works had been made. The other two Papers referred to towns which were situated on the sea-board. All three Papers were alike in one respect, that they described works involving sewage-treatment—he would not say in its more advanced form, but certainly up to the standard of many inland towns. He asked why all that extra purification had been adopted in two coastal towns. It had been said that certain profits were to be obtained from the products of sewage, but he did not think that a corporation would get very rich on profits derived from sewage-products. The question was why, with the ocean alongside, had not the ocean been chosen for depositing the sewage, with or without ordinary sedimentation. Mr. O'Shaughnessy had given an interesting account of what was done at Birmingham, where natural conditions demanded that the effluent should be purified to a very high degree; but with towns situated near the ocean it should be very carefully considered whether a sewage-installation should not be so laid out that difficulties would not arise in the future in the event of its being found desirable to turn the whole of the sewage-product into the sea. At Colombo there were two outfalls, one about 2 miles up the Kelani river and another on the seaboard. That on the sea-board, it was true, presented certain difficulties in construction, and it was impossible to dogmatize in connection with places, because in every place conditions had to be carefully considered; but it did not appear to him that there, as at Capetown, anything further than turning the sewage or the effluent into the ocean was necessary.

Mr. Watson. Mr. G. WATSON remarked, with regard to Mr. Tickell's Paper, that other sanitary improvements had been made during the period

Mr. Watson. under review. For instance, the refuse of Colombo was now burned in a destructor for which he himself had had some responsibility, and that, no doubt, had helped to reduce the death-rate. He thought Papers from the Colonies on the subject of sewage-disposal were welcome; and, in criticizing them, it should be remembered that the Authors had not all the advantages of home members in keeping their knowledge up to date. Mr. Jameson described one or two methods which were comparatively novel. It was quite evident that he had been dealing with a very difficult problem, having such a large quantity of pail-contents as well as sewage; and his method of putting it into sewers and treating it had evidently resulted in considerable saving to the town. The treatment he adopted for making it into a fertilizer appeared to be worthy of attention. By drying the sludge sufficiently, grinding it, and mixing it with enriching material he obtained a saleable manure—evidently a profitable operation, otherwise it might be assumed that he would not go on with it. It would appear that the use of sludge as a fertilizer was not sufficiently advanced in England. At the present time sludge was being treated and sold as manure in Birmingham, and he believed that very soon it would be found generally to be a paying proposition. The draining and drying of sludge on a bed of town refuse, which Mr. Jameson was attempting, seemed to be an interesting combination, and would be of great importance in the future. The disposal of street-sweepings, town refuse, and sewage-sludge in combination seemed to offer great commercial and financial advantages over dealing with them separately.

Mr. Tickell. **Mr. Tickell**, in reply, thanked the meeting for their reception of his Paper. No unusual methods had been adopted in the excavation in running silt. Close timbering was put in, and the runners were kept well driven down before any silt was removed. Underdrains were put in, and the sumps were kept clear of the main trenches. Pulsometer or other pumps were installed. By careful workmanship, and with the help of excellent foremen from England, accustomed to dealing with deep excavation in all ground, there were very few mishaps. With regard to intercepting-traps, he had been concerned with the construction department, and on completion the work had been handed over to the municipality, who put in the house-connections. He had helped them in drafting the drainage by-laws, and had put in a clause by which the intercepting-trap was compulsory; but subsequently he gathered that they either had abandoned or intended to abandon it. He had no very strong views on the subject, but he thought the use of the trap was a precaution which would probably exclude rats—a very important thing

Mr. Tickell. in an oriental country which was liable to plague, as the rat-flea was the great carrier of plague. Mr. Humphreys had also asked about the necessity for treatment. In the original scheme for Colombo it was intended to run the sewage out to the mouth of the river and to have it carried out to sea without treatment, or to discharge it only on the ebb tide. That had to be abandoned owing to the proposed construction of a wet dock to the north of the town, which would have cut through the line of the main sewer. Consequently it was necessary to make an alteration in that line, and a sea outfall could not be obtained, because the sewage could not go into the harbour. The river was used for religious bathing purposes and there was a great outcry from sections of the community on the sewage being put into it. From consequent analyses, however, they seemed to have come to the conclusion that the water a little below the outfall was purer than it was above. That was due, no doubt, partly to the water coming up with the tide. In the evolution of the drainage-scheme the southern outfall was subsequently adopted, owing to the extension of the municipal limits, and in that case there was no alternative to the sea outfall. There was a reef about 300 yards beyond the shore, so that sewage was discharged into a sort of lagoon or backwater. The currents varied, being sometimes from north to south and sometimes south to north, but anything discharged into the sea there was liable to come up on the beach, and treatment was therefore necessary, although not a high degree of purification. There was no stream running towards the south of the city which would have provided any current to carry away objectionable matter. He did not wish to give the whole credit for the health improvement to the drainage-scheme, because other sanitary measures had been adopted which had helped to reduce the death-rate. He calculated that the scheme really paid about 10 per cent. on the outlay owing to reduction of sickness, and in that calculation he took no account of the saving of medical expenses, which he had balanced against the savings due to the other sanitary measures.

⁂ Mr. Lloyd-Davies's reply will be found at p. 148: Mr. Jameson's reply (if any) will be printed in the next volume of Proceedings.—SEC. INST. C.E.

Correspondence.

Mr. M. R. Atkins, having been associated with the carrying out of the works at Colombo from 1906 to 1919, and having been concerned subsequently with drainage-works in Calcutta, wished to remark on the relative advantages and disadvantages of the separate and combined systems as applied to those two cities. In Colombo, with a separate system of underground rain-water drains to which all street gullies and surface drains were connected, very careful scheming was necessary to secure sufficient differences of level, between the sewers and rain-water drains and their respective connections, to allow of their crossing and recrossing each other at street-junctions and elsewhere without break of gradient. In many cases the invert-levels of the rain-water drains were below these of the branch sewers; that fact made it possible for the upper lengths of the rain-water drains to be dispensed with, and for the branch sewers to be used as combined drains, which were made to overflow automatically into the rain-water drains at any desired rate of flow without "heading up." The chief disadvantage of that arrangement was the fouling of the air in the rain-water drains by sewer-gas passing through the overflows; but that was obviated by fitting the latter with flap valves. The advantages of the arrangement were its economy and the convenience of having one sewer instead of two in the narrow streets where it was found possible to apply it. The combined system which was in use in Calcutta avoided the complications previously referred to and was well suited to the needs and habits of the population; but it failed to prevent flooding of the streets whenever the rainfall exceeded about ½ inch per hour. It was argued that the rapid flushing-out of the sewers which occurred on such occasions was beneficial, but the system was at best only a compromise, and did not deal with the problem of a tropical rainfall in the comprehensive spirit which modern engineering practice demanded. An interesting adjunct to the sewage-treatment at Colombo, which was not mentioned by Mr. Tickell, was the treatment on experimental drying-beds of septicized sludge from the circular sedimentation-tanks. The beds had concrete floors, with central drainage-channels, and brick walls 3 feet high, so as to provide depth for a 6-inch layer of bricks or drainage-tiles, 12 inches of filtering medium, and 12 to 18 inches of liquid sludge. Different sizes of broken stone had been tried for Mr. Atkins.

Mr. Atkins. the filtering medium, but the best results had been obtained with a 9-inch layer of 1½-inch stone, topped by 3 inches of ¾-inch stone. When the sludge was first run on there was a slow trickle of dirty water through the central drain, but within 10 minutes all trace of discoloration disappeared, and the water became perfectly clear. The sludge settled down to about one-third of its depth as it dried, and was ready for spading within 10 to 14 days, according to the weather. Examination of the beds showed that the sludge never penetrated more than 2 inches below the surface of the filtering material, and cleansing of the bed was effected by removing and washing the top layer of stones without disturbing the remainder. In order to avoid having to wash the stone, Mr. Atkins had tried replacing the 3-inch layer of ¾-inch stone by a layer of coco-nut matting weighted down by flat steel bars. The matting was quite effective in arresting the liquid sludge and could be taken up and cleaned in a few minutes after the dried sludge had been removed by spading. The surface of the stone under the matting was not even discoloured, and the matting showed no sign of rotting after being in use for several months.

Mr. Braine. Mr. C. D. H. **Braine** confirmed Mr. Jameson's statement that there was no nuisance from sewer-gas. The mention of dosing-tanks in the description of the outfall works suggested chemical treatment; but there was no such treatment. The dosing-tanks were shown in *Fig. 1* (p. 87) as distributing-tanks for regulating the flow to the oxidizing-beds. The Aapies river had its source at some large dolomite springs, which were used for the Pretoria water-supply. The Water Court had lately decided that, until permanent gauges had been erected, the combined flow was to be taken as 6,178,000 gallons per diem. The residual flow in the river was extremely small until it had been increased by bath-water, sewage-effluent, etc. The acreage of land available for sludge-drying and for the treatment of the septic effluent was far too small, especially as the sludge was no longer used for making fertilizers. The additional area of 2½ acres required during the hot weather was due to the fact that the summer was the wet season. The attempt to irrigate growing crops with the septic effluent had been unsuccessful. The land used for the treatment of the sludge and septic effluent yielded huge crops of tomatoes, etc., which were self-sown by the sewage. These, however, were not marketed. At the present time a population of 63,000 was served by sewers and the pail system. The latter applied to about 6,000 houses, and about 3,000 pails were tipped at the two collecting-tanks each night. The tanks were 30 feet square, and about 4 feet deep, with a large screened outlet in the centre. A vertical screen surrounding the outlet was far more effective, and gave less

Mr. Braine.

trouble with clogging, than did a horizontal screen covering the outlet. All nuisance was practically prevented by discharging the pails under water. The cleanliness at the collecting-tanks was amazing. During the day there was not a taint in the air. The tanks were about a mile from the centre of the town, whereas the original depositing-ground was 8 miles away; which gave some idea of the saving under the present system. The walls of the oxidizing-beds consisted of dry-stone work, and the smallest medium used passed through a ¾-inch screen, anything passing a ½-inch screen being rejected. The sprinkler arms were 80 feet in diameter, and worked under a head of 27 inches. They had been running for 10 years and were still in good condition. Dry grass and weeds, etc., were burned on the surface of the septic tanks and the drying sludge in order to deal with the plague of flies. The deposit of ashes was very effective but it retarded the drying. The tanks were now being roofed with galvanized iron to keep out the rain, and were also being made fly-proof. As there would be ventilation through the wire screens, the heat under the roof might result in more rapid drying. Quite a surprising quantity of sand, ashes, cinders, grape-seeds, etc., was removed from the effluent channels leading from the septic tanks. An experiment had lately been made of spreading 50 tons per acre of sludge, containing 76 per cent. of moisture, on cultivated land, and treating that with 1,500 lbs. of agricultural lime per acre. That has resulted in the rapid humification of the sludge.

The history of the drainage of the southern suburbs of Capetown was of interest. Twenty years ago, Woodstock, Mowbray, Rondebosch, and Wynberg were independent municipalities, employing their own engineering staffs, and carrying out more or less isolated programmes of work. During 1903–4 he was engaged on the preparation of plans for the drainage of Mowbray, although it was perfectly obvious that nothing short of co-operation with the adjoining municipalities could result in an economical scheme. It was then found that a gravitation scheme was impracticable, and that the only solution was on the lines carried out by Mr. Lloyd-Davies. The isolated programmes included water-supply. The questions became so acute that the Government appointed the "Peninsular Commission" to inquire into, and report on, the unification of the suburban municipalities. In 1913, the Provincial Council passed a Unification Ordinance uniting Capetown with other municipalities. The cost per head for sewerage-works and water-supply in the residential suburbs of South African towns was always high, on account of the scattered positions of many of the houses. That applied to the Cape suburbs, where a large number of the houses were

Mr. Braine. surrounded by extensive grounds. In the Johannesburg suburbs building-plots more than an acre in extent were common, and some of the gardens included more than one plot. In at least one case the total area was about 5 acres. That meant that the length of sewers per house was vastly greater than under ordinary European conditions. The Liesbeek and Mowbray-Woodstock intercepting sewers ran practically at the foot of Table Mountain slopes—an outlier of this mountain was shown in Fig. 1, Plate 1. The rainfall over the area served varied greatly. At Maclear's Beacon on Table Mountain (3,478 feet) the average rainfall was about 87 inches per annum, at the Royal Observatory (near the Maitland intercepting sewer) it was about 20·74, and at Newlands about 54·65, with a maximum of about 70 inches. It frequently rained in Rondebosch when the weather on the Camp Ground was clear and bright. The wet season was in the winter months—May to October. The ratio of the thickness of the walls to the diameter of the culverts for the "larger sections" was given; but there was no mention of the diameter above which the ratio applied. He agreed that the correct procedure was to graduate the sizes of the sewers to receive storm-water according to rainfall-intensity, and it would have been interesting if the calculations had been given for a definite length of sewer. Unfortunately, no longitudinal section showing the dimensions of a sewer was given in the Paper. For the benefit of engineers living in places where they had to limit their own libraries, and where reference-libraries were not available, he suggested that it would be as well to give the formula used for calculating the sewer-capacities. Details for the numerous Liesbeek river-crossings would be of interest. The use of brushwood in the bacteria-beds was a new departure for South Africa. The Cape Flats had been largely reclaimed by planting Australian wattles,[1] and bush should be cheap; but it seemed doubtful if that method would result in lower maintenance-costs. Useful information might also be given concerning the crops irrigated by the effluent from the humus-tanks, and the profits obtained from the farming-operations.

Mr. Tickell's Paper was an interesting account of a large and important piece of work, which seemed to bristle with difficulties of construction; but how these had been overcome was not explained. It was not easy to follow the Paper, for landmarks were referred to that were not shown on the plan; and while reduced levels were given, there were no sections. In the harbour area there was a trench 43 feet deep and a sewer constructed in liquid peat mud. Mr. Braine could not help wondering if there was no way round, and no possibility of arranging the pumping-lift so as to carry the sewer over the swamp. No description was given of the Kelani river, and it would be interesting to know why the outfall of the sewer was moved 2 miles upstream. Were the two 48-inch cast-iron pipes found to be cheaper than concrete culverts? With so much work in running sand and near rickety buildings Mr. Tickell must have had some valuable experience that would be highly useful to other engineers. It was to be hoped that he would give some information on those points and also say if any form of cementation process had been tried. It would be interesting also to hear more about the construction of the sewer under two bays of the lake. Special precautions must have been required to prevent the flotation of the sewers. The southern outfall merited further information. A coffer-dam to withstand the full force of the south-west monsoons must have been of very substantial construction. Apparently only the effluent was discharged into the sea. The reduction in the death-rate following the construction of the works was useful information and highly satisfactory; but it was not surprising to anyone who had lived in the Far East and had experienced the reek from stagnating pools of filth. The saving in funeral-expenses was, he thought, a novel and interesting point. Mr. Tickell was to be congratulated on having carried so large a work to a successful conclusion. Mr. Braine.

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[1] Minutes of Proceedings Inst. C.E., vol. cl, p. 377.

Mr. Camm. Mr. **Edward Camm** was interested in the procedure for designing sewers to receive storm-water. When preparing a drainage-scheme for Accra, Gold Coast, in 1920, he had followed a method similar to that described by Mr. Lloyd-Davies, based on the inverse ratios of the drainage-area and the rainfall-intensity. He had found that previous schemes had adopted rainfall-rates of 5 inches and 2½ inches per hour over the whole of the districts respectively dealt with. The rainfall-intensity curve he had selected as best meeting the observed rates of rainfall at Accra was represented by the equation $(T + 18)\,R = 135$. That curve gave about double the values of the Capetown curve, namely, for areas of 1 or 2 acres, a rate of 5 or 6 inches, and on an area of about 700 acres, a rate of 2¾ inches per hour. The frequency of such high rates was a matter requiring careful consideration. In England rainfall-intensities of 4 and 6 inches per hour had been recorded for short periods; but it might not be an economical procedure to make provision for heavy falls which only occurred at rare intervals. On the West Coast of Africa, however, such intensities had been recorded several times in a year, and consequently drains of large size had to be constructed. In Mr. Lloyd-Davies's scheme the storm-water drainage-area was limited to 100 square feet per house. Special precautions would be necessary to make that limitation effective. Mr. Camm

Mr. Camm. had had similar restrictions in view when preparing a drainage-scheme for Port Elizabeth; but, owing to the topography of the town, it was decided to take in all the back roofs and yards; the actual area per head, varying with the density of population, was calculated from typical areas. He had expected to see a larger provision for rain-water in the sewers of the Colombo Drainage Works; but he assumed that the figures given, which varied with the proportion of built-up area, included allowances for evaporation and absorption. In the case of Capetown the whole of the rainfall on the 100 square feet per house appeared to be taken as reaching the sewers. At Accra discharges ranging from 8 to 50 per cent., according to the development of the district, had been adopted.

Mr. Mager. Mr. F. W. MAGER observed that, being responsible to the Government for the drainage of several towns in the Federated Malay States, all of which were on the surface system, he found Mr. Tickell's Paper, describing the conversion of the former surface-drainage system of Colombo to the underground system, of considerable interest. The climate, rainfall, and other conditions of Colombo approximated very closely to those of Malaya, and the general problem was the same in all material respects. The dry-weather flow for Colombo was taken at 25 gallons per head of population per 24 hours, and the sewers were designed to take six times that quantity. Twenty-five gallons per head appeared to be low. The standard allowance for town water-supplies in the Federated Malay States was 40 gallons per head, and the maximum daily consumption exceeded the average by 50 per cent.; that, if applicable to Colombo, would leave 25 by 4½ = 112·5 gallons per head allowance for storm-water, giving 62·5 cubic feet per second discharge to the underground sewers from an area of 9 square miles. The volume in cubic feet per second of the discharge from catchment-areas in Malaya had been found to be as much as $500\ M \times \frac{4}{5}$, where M denoted the area in square miles: for Colombo that would give 2,900 cusecs, so that 2·1 per cent. of the storm-water would be admitted to the sewers. Hence the provision made for storm-water seemed low, even for unpaved areas like the Cinnamon gardens. In the denser paved areas such as the Fort or Pettah quarters, the population was 183 per acre, and the provision for storm-water per acre would be 0·038 cusec. A rainfall of 5·5 inches per hour on a paved area might appear wholly as discharge, and in that case 94 per cent. of the storm-water would be excluded from the underground sewers. But it was stated that only 2 inches per hour was provided for by the rain-water drainage-system; therefore it appeared that there was a large excess of storm-water unprovided for. Consequently a real danger existed of damage by floods to property

Mr. Mager.

generally and, by overcharging, to the underground sewers themselves. Experience, however, might have proved these forebodings to be groundless. He would like to have information on this important point. Further information on the subject of house-connections, both in the residential quarters and in the bazaar, would be of interest. Modern sanitary fittings were almost unknown in Malaya, and the results of their use, or rather the neglect of their use, by native servants in better-class residences—and still more by the inhabitants of the native bazaars—was a matter of conjecture. He gathered from what was stated in the last paragraph on p. 78, that the pail system, with all its attendant disadvantages and danger to health, was still in existence over a considerable area. If that were so, then those areas must be little improved from a sanitary point of view, the large expenditure of Rs. 18,000,000 being scarcely justified. The reduction of 18 per cent. in the death-rate might be due to some other cause or causes than the construction of the interesting works described in the Paper.

Mr. McConnel.

Mr. SAMUEL MCCONNEL observed that probably no subject in the whole field of sewage-disposal had aroused more controversy in the past few years than that of settling-tank treatment. The necessity for early separation of the settling and putrefying processes was generally admitted, but the methods of doing this were many. The rectangular flat-bottomed settling-tanks adopted in practically all large English towns had proved successful and economical, sludge-removal taking place every fortnight or so; but in countries where higher temperatures prevailed, bacterial action was accelerated, and sludge must be removed very frequently, or simple settlement would be seriously jeopardized. This rendered advisable some form of tank so designed that sludge might be removed at short intervals. Tanks of the Dortmund type, as used in Pretoria, had many advantages over the double-storey Emscher type. There was no possibility of the sludge interfering with the settling process; and, where the ground was bad and waterlogged, it was more economical to reduce the depth of the tank and to find capacity for sludge-storage elsewhere. The slope of the bottoms of the Cape Town tanks appeared to be very slight; some means of agitating the settled sludge preparatory to drawing it off would be necessary Settling-tanks similar to those in use at Pretoria could be seen at Brightlingsea treating crude sewage, Skegness also employed this type of tank, but the sludge was drawn off continuously. That suitable tanks could be designed without using excessive depths was shown by the reconstructed rectangular tanks at Colombo. The Emscher tank had gained little popularity in Great Britain, where separate sludge-digestion tanks were almost universally employed. Storm-water tanks, ably advocated by

Mr. McConnel. Mr. H. P. Raikes, M. Inst. C.E., satisfactorily displaced storm-water treatment-beds, which involved great expense in construction, and, owing to their irregular use, were not particularly efficient. The Capetown flats appeared to be an ideal site for sewage-farming, the conditions being very similar to those prevailing in Melbourne, where settled sewage was treated directly on the land without the use of percolating filters, etc. Probably there were local reasons why similar treatment would not be suitable at Capetown. The precast pipes used for sewers were made by the wet process, in moulds. A machine of Scandinavian manufacture was now in use for constructing pipes of a dry mixture, mechanically tamped, which obviated the use of forms. Such pipes could be made with or without sockets very cheaply and quickly, and had displaced the wet cast pipe in many places. In calculating the discharge of sewers, pipes, etc., much labour might be saved by the use of the Hazen-Williams slide-rule based on the well-known formula. By means of that simple instrument the use of logarithms, tables, etc., was entirely obviated.

Mr. Olive. Mr. W. T. Olive thought that the local authority had erred in abandoning his proposal for a gravitation scheme of sewerage for the Southern suburbs, with a sea discharge 4¼ miles to the north-east of the sea outfall at Green Point. A similar scheme was carried out by him 27 years ago, as City Surveyor, for old Capetown, and had been in satisfactory use for the last 25 years. It was decided upon after exhaustive examination of all available professional opinion, and of a marine survey made expressly to determine the average resultant trend of the currents of Table Bay, the object being to obviate all treatment-works. Had that proposal for the suburbs been adopted, considerable economy would have been effected, inasmuch as less than one-tenth of the sewage would have had to be lifted, whilst the cost of pumping and treatment of the whole quantity involved in the scheme under discussion would have been eliminated altogether. The same extravagant expenditure could be illustrated, in a minor degree, by reference to Wynberg which he also advised, when called in as consulting engineer, should be given a sea outfall by gravitation. But, in spite of warning, that municipality persisted in having its treatment-works, which in a few years after completion had to be abandoned. The sewage now was carried 5 miles farther into the country, and new works had been built, at further expenditure. That was common knowledge before the Capetown Corporation decided to favour a pumping scheme and sewage-treatment for the southern suburbs. Reverting to the proposal for dealing by gravitation with the whole sewage from the southern suburbs, including Wynberg, the following figures gave the invert-levels, at the leading points, of sewers which

Mr. Olive.

would take the sewage from the several suburbs into a joint trunk main having its outlet into the ocean :—

	Above L.W.O.S.T. Ft.
For Wynberg	65
„ Claremont	33
„ Rondebosch	20·5
„ Mowbray	12·9

In no case would the falls of those intended sewers reach the following minimum permissible gradients :—

Diameter : Inches	4	5	6	7	8	9	10	12	15	18
Fall : 1 in	70	90	150	200	240	270	300	350	420	500

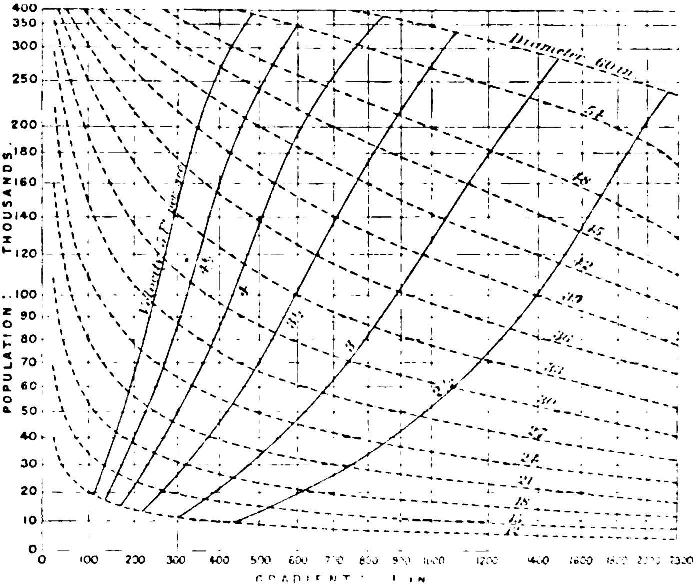
Diameter : Inches	21	24	27	30	33	36	42	48	54
Fall : 1 in	600	700	800	880	1,000	1,050	1,200	1,310	1,550

A water-supply of 25 gallons per head, with one-half in 400 minutes, was taken as a maximum rate. A rainfall-admission of 45 gallons per head was equivalent to a yard-surface of 23·4 square feet at 1 inch per hour. These values for sewage and rain worked out at 0·01 cubic foot per minute per head of prospective population. *Fig. 1* (p. 138) showed the sizes of sewers, as flowing three-quarters full, required for the calculated populations in the respective districts and sub-districts to the gradient available therein.

He regarded the tipping of pail-contents into the main sewers at Pretoria as a reprehensible and retrograde practice. It was almost impossible to dilute it by so-called flushing—certainly impossible with the 20 gallons per pail mentioned. Most modern water-closets received a 3-gallon flush at each operation, a dilution of about 150 times. From the data given the water-consumption was 37 gallons per head per day, so that the solids produced individually were diluted fully 1,480 times, even counting all the population as adults. Moreover, deposits inevitably occurred in the main sewers receiving the tipped stuff, which fermented and fouled the system. The oxidizing-beds dealt with 150 gallons per square yard per diem. As the dry-weather flow was 1,500,000 gallons, there should be 10,000 square yards of beds: in effect there appeared to be 2,240 square yards provided in the four (*Fig. 2*, p. 89). Consequently he concluded that the figure was intended to show only one-quarter of the works, and that the screens, grit-chambers, distributing-tanks, etc., were all quadrupled. The manurial value of sludge had been discovered, but could Mr. Jameson give any information as

Mr. Olive. to the quantity sold and the price realized per ton of dry sludge, and as to the price obtained after adding phosphates?

Fig. 1.



Sewers and Gradients Relation to Population (Sewers running Three-Quarters Full).

Mr. Roechling. Mr. H. Alfred Roechling observed that Mr. Jameson used some words and terms which were not clear to him, e.g., "elongated horizontal flow." It was stated that "crude sewage might be primarily separated from organic matter." To what process and to what stage of the organic matter did Mr. Jameson refer? Mr. Roechling had never heard of a process in which separation of the major from the minor portion of sewage was attempted. He assumed that the term "sweet sewage" meant fresh sewage. On p. 93 Mr. Jameson used the expression "molecule of liquid sewage." Did he do so in a strictly physical or chemical sense, or did he merely wish to indicate minute particles of sewage? On the same page he mentioned "colloidal matter or humus," and this seemed to indicate that colloidal matter and humus were synonymous terms. Such a conception was not adopted in this country. It was quite true that the meaning of the word "colloid" had undergone some slight change, but not in the direction in which Mr. Jameson apparently used it. Colloids formed the domain that lay above molecular and below macroscopic dimensions. Although humus might consist of colloids

Mr. Roechling.

and other matters, it could not be said that colloids were humus and vice versa. In sewage colloids were partly-dissolved matters existing in a state of pseudo-solution, together with matters in emulsion. In true solutions the particles had been split up into their molecules, but in colloids the disintegration had not yet proceeded so far, and it was therefore a characteristic of colloids to render a liquid turbid (colloids mainly in the "hydrogel" or "gel" state) or to give it a mucilaginous or opalescent character (colloids entirely in the "hydrosol" or "sol" state). Colloids were easily subject to changes both in the direction of true solutions and in that of undissolved substances, the former change being brought about by anaerobes in putrefaction, the latter by aerobes in a biological-treatment plant. In town-sewage colloids were chiefly of an organic nature, mostly sulphurous and nitrogenous compounds, whereas in factory-wastes inorganic colloids such as iron salts might also be present. Both the reversible and the irreversible kinds of colloids were present in sewage, and the organic colloids were the carriers of putrefaction and the cause of the water-retentive powers of sludge. It was probably correct to say that, after the removal of the suspended matters by preliminary treatment, 50 per cent. of the remaining organic matter to be dealt with in a filtration-area was in a colloidal state. He assumed that the expression "functional response of self-cleansing flow" (p. 94) meant the effect of a self-cleansing velocity. On the same page it was stated that the humus contents of a percolating-bed were driven out "ahead of the flush." Were the humus contents driven out by air or by the liquid with which the bed was dosed? If the air were driven out in front of the liquid, at a velocity sufficient to cause the humus to be removed, the rush of air must be very considerable to remove the material caked on the medium of the bed. It was desirable to preserve the air in the bed as far as possible, and not drive it out by sudden rushes of the liquid. He was afraid he could not agree with the theory of evolution advanced by Mr. Jameson. What had been described was rather retrogression than progressive development, and dealt with only a portion of the question. From a purely physical standpoint, sewage consisted of the liquid and the various matters suspended and dissolved therein, and three distinct stages in artificial sewage-treatment were generally recognized, namely, the removal of the suspended matters, the purification of the liquid, and the disposal of the sludge. Mr. Jameson dealt only with the first stage, and, further, he omitted any reference to the activated-sludge process, which probably formed the latest stage of the artificial treatment of sewage. The Cameron septic tank was only a revival, on a larger scale and under a different

Mr. Roechling. name, of the "cesspit." Hence that tank had now ceased to be employed for sewage-treatment. The Travis tank and the Imhoff or Emscher tank—the latter being to all intents and purposes a Travis tank with some slight modifications, which did not affect the principle and which were no improvement—stood upon a different footing altogether; but they represented only a modification of the apparatus used and not the principle of the general treatment. Mr. Jameson's third stage was again a retrogressive step, a reversion to an old type previously used, and, to some extent, abandoned again. To call the tank a "Watson-Dortmund" tank was a complete misnomer. The tank was a common vertical tank, such as was used for the first time for sewage-treatment in Halle a/S in 1886. It was then slightly altered by Mr. Kniebuehler, of Dortmund, who introduced about 1888 a spreader at the outlet of the inflow-pipe; afterwards it was used in other parts of Germany, generally without Kniebuehler's spreader. But it did not prove of much advantage over horizontal rectangular tanks, first used in this country, so that its higher cost—especially where large quantities of subsoil water were met with—was not justified, and in this country it had never been employed on a large scale. One of the reasons urged against it was the percentage of water—about 95 per cent.—contained in its sludge, as against about 90 per cent. contained in the sludge of horizontal tanks. Some 30 years after its invention Mr. Jameson had used this tank for the very purpose for which it had been originally intended, so that even his fourth stage of evolution foreshadowed for South African practice was non-existent. Mr. Roechling regretted that the diagrams given in the Paper were not drawn to scale. Were the analyses of sewage and effluent those of chance samples only, or were they averages of collective samples taken over an extended period? He regretted also that no figures of the capital or working costs of the plant had been given. From a Government report he inferred that Pretoria had spent about £175,000 on storm-water drainage and about £300,000 on the sewerage of parts of the town, which figure, he surmised, included the cost of the sewage-treatment works, or a total of about £475,000. The total municipal debt at the end of 1920 was about £1,545,287. If those figures were correct it followed from the figures given in the Paper that the cost per head of population of the sewered area was nearly £16 for storm-water and sewerage, and about £10 for sewerage only. These were high figures. The volume of the dry-weather flow of sewage was about 50 gallons per head, and taking 30 gallons as an ample allowance for the water-supply per head, 20 gallons were left to be accounted for. He thought that additional quantity was

Mr. Roechling.

subsoil water which found its way into the sewers through leaky pipe-joints. The subsoil under Pretoria was saturated with subsoil water, and the analysis of the dry-weather flow of sewage confirmed his view as to this influx. The sewage was weak, and the late Royal Commission remarked on such sewage that, as a rule, it contained much subsoil water. Another point of confirmation of this view was the flow of water in the main sewer between 2 a.m. and 5 a.m., which he estimated, from the particulars given in the Paper, at 36,000 gallons per hour. In his view the subsoil water should have been excluded from the sewers when they were laid, as that precaution would have saved extra expense in treatment. The omission of the intercepting trap to house-drains was a mistake. If there was nothing to be removed from the street-sewers, why should they be ventilated through the house-drains ? The flow of air in house-drains could not be regulated; it was very often in a downward direction towards the street-sewer. A Departmental Committee of the Local Government Board (now the Ministry of Health), came in 1911 to the conclusion that the free ventilation of sewers appeared to be unnecessary. It was, however, quite a mistake to assume that no sewer-gas was produced in public sewers, as experience has proved that, however well a sewerage system was designed, carried out, and maintained, nobody could control either accidents at the time of the minimum flow, through substances getting into the sewers, or accidents to the sewers themselves. A disconnecting trap, properly designed, fixed, and maintained, need not become blocked; where blocking did occur, the fault was generally a local one which could easily be remedied. Although it had not been possible up to the present to establish a clear connection between sewer-gas and some epidemic diseases, the methods of investigation were still somewhat crude. It seemed not impossible that, with more refined tests, specific disease-exciters or volatile ptomaines might be discovered in sewer air; and in matters of health it was very unwise to sail too close to the wind, as prevention was far better than cure, even if cure was at all possible. A great deal had been made, by the opponents of the intercepting trap, of the Departmental Report previously referred to; but the view of it taken by the Ministry of Health was quite conclusive in this respect. Although that Report was made in 1911, the Ministry of Health had issued, since July, 1921, model by-laws, in which the intercepting trap was still maintained, and all that they had done was to give local authorities on their application a provisional option to omit the disconnecting trap, where the house-drain was connected with a street-sewer; but where the house-drain was connected with a cesspit the Ministry still insisted on the trap. Mr. Jameson

Mr. Roechling. employed 10 acres of land for the treatment of the effluent from the sludge-digesting tanks, and 25 acres which were cultivated and presumably treated with some portion of the effluent from the percolating-beds. He hoped Mr. Jameson would make that point clear. There was apparently no effluent from the land treatment. It was not stated what quantities of floating and suspended matters were arrested at the screens and in the detritus-tanks, but he thought that the matters retained in the latter could only be of trifling amount, owing to their form and the short sojourn of the sewage in them, although it was stated that the sewage was practically quiescent in the centre. This view was confirmed by the short time occupied in cleaning the detritus-tanks.

The work done by the vertical settling-tanks was disappointingly small, the retention of suspended matters being only about 50 per cent. Horizontal tanks in this country and on the Continent frequently retained 80–90 per cent. of the suspended matters, and produced sludge with only 90 per cent. of water, whereas the Pretoria sludge had 96 per cent. of water. In vertical tanks the water-content of the sludge was frequently higher than in horizontal tanks, and had the latter been constructed, the great expense of deep foundations in water-logged subsoil would have been saved, and the quantity of wet sludge might have been reduced by about 38 per cent., which would have been equivalent to a reduction of nearly 85 per cent. in the work of the percolating-beds. He was of opinion that the effluent from the settling-tank could no longer be called fresh sewage, as during its passage through the sewers and its sojourn in the tank it must have become septicized to a considerable extent. In this country sewage became septic on an average after about 9 hours, and in the great heat of Pretoria the time must be much shorter. That view was confirmed by Mr. Jameson's remarks about the septicity of the pail-contents and the rapidity with which matters retained in the detritus-tank and vertical settling-tank became septic.

Mr. Jameson stated that 1 square yard of the percolating-beds dealt with 150 gallons in 24 hours, but Mr. Roechling found that, with only twelve beds at work, the rate was 224 gallons per square yard, equivalent to 112 gallons per cubic yard. Mr. Jameson had stated further that by using only twelve of sixteen percolating-beds, with four as a stand-by, the beds would have an almost indefinite life, that no difficulties with humus-like material had been experienced, and that there was no need to extract this matter from the effluent in special tanks. Mr. Roechling found some difficulty in accepting these statements, and, unless the work done by the filters at Pretoria was entirely different from what it was in Europe, he felt sure that sooner or later they would become choked. Those beds received about 16 tons of wet sludge per million gallons; and as only about 2 tons was left in the final effluent, it was clear that about 14 tons was deposited in them, which was equal to 21 tons per day. If, as it appeared, each bed was worked continuously for say, 6 months, each cubic yard of filtering material received about ¼ ton of wet sludge in that time, which in volume was equal to about ⅓ cubic yard. That quantity was approximately equal to the pore-volume of a cubic yard of the filtering-material, so that at the end of 6 months the percolating-bed would be completely filled, to the exclusion of air, unless in the meantime a continual removal of the retained matters had taken place. In addition, it was quite possible that fungoid or other growths formed on the surface and these might cause further difficulties. They had experience at Pretoria of the clogging of filters when dealing with night-soil sewage. If no humus-like material was found in the effluent, the matters brought on to the beds must have remained in them, and that view, was supported by the statement that after, say, 6 months of continuous work each bed had to be given a rest of about 2 months, during which the self-purification processes would be at work. Mr. Jameson mentioned the Hampton theory, with which he appeared to agree, and it was an essential point in that doctrine that the pseudo-dissolved solids, deposited in the filters as solid matter, remained and increased in them, thus increasing the absorbing area and the efficiency of the filters, until they became choked. The view that the purifying effect of percolating filters was chiefly brought about by bacteria was no longer tenable; their participation in this work seemed only to be indirect, and recently the view had been expressed that practically all the pseudo-dissolved solids were retained in the filters purely by mechanical action, leading eventually to choking. The work done in the Pretoria filters formed an interesting subject, and he trusted Mr. Jameson would throw further light upon it in his reply. Nothing was said about nuisances from smell and flies from the filters, but he could not help thinking that, under the Pretoria conditions, such nuisances must be very pronounced at times. Mr. Roechling.

Mr. Jameson mentioned the standard of purity for effluents laid down by the late Royal Commission on Sewage Disposal, and stated that no difficulty was experienced in complying with it; but Mr. Roechling thought that statement was hardly correct. It was not clear from the analyses whether the 5-days dissolved-oxygen test had been made on the whole effluent, including the suspended matters, as insisted on by the Royal Commission. As to the suspended matters, the Pretoria effluent was only just within the limit, but by the 5-days dissolved-oxygen test it did not comply with the standard. Further, Mr. Roechling. the standard of the Royal Commission required eight dilutions in the stream which took the effluent, and, as in the Aapies it only received three dilutions, the requirements of the Commission for a more stringent standard should be applied. He regretted that nothing had been said about the condition of the Aapies below the works, and he would be glad to know whether any of the usual flora associated with sewage-effluents had been started in it, or whether at Pretoria a different flora had made its appearance.

The experiments in sludge-treatment at Pretoria had been made over and over again elsewhere. From the figures given in the Paper he calculated that each sludge-digesting tank received about 4,000 gallons of liquor in 24 hours, containing about 31 per cent. of wet sludge with 96 per cent. of water, and 69 per cent. of septic sewage. The tank held about 23½ days' flow, but, as the outlet was now left open permanently, it could only contain in addition to the same, so far as he could estimate from the information in the Paper, about 6 days' flow. That meant that, on the one hand, the liquid and its sludge contents were only exposed for 6 days to septic action in the tank, and that, on the other hand, every day at least 4,000 gallons of very foul liquid escaped from the tank and had to be purified on land. If 6 days were sufficient for the formation of a thick scum, it showed how very rapid the putrefactive changes were at Pretoria; but he thought it was probably due to the method of working that it took a whole year to reduce the water-content of the upper layers in the tank from 96 to 76 per cent. He was surprised to learn that it took 1 to 2 months to dry the sludge on the land, and he thought that could only be due to the large amount of colloids it still contained. It was said that in the Imhoff tank the water content of the sludge was reduced in about 6 months from about 95 to 75 per cent., and that after removal from the tank the sludge took only some 10 days to dry on land. It would be interesting to know what became of the gas formed in the tanks, which he estimated at 5,000 to 10,000 cubic feet daily. Without the formation of gas the sludge could not rise from the bottom; and, if a thick scum were formed, the gas would have to force its way through to escape. If it had been necessary to discharge the liquid from the sludge-digesting beds into the Aapies, no doubt great difficulties would have been experienced before now, and it was only owing to that convenient back door of land-treatment that the difficulty had been avoided so far. The land did the heaviest work at Pretoria.

He missed from the Paper a justification for the adoption of an artificial treatment of the sewage. South Africa was mainly an agricultural country, where every drop of liquid was required on the

Mr. Roechling.

land, and he thought it was the duty of the designer of a sewage-treatment plant in such a country to adopt land treatment, unless there were absolutely compelling reasons against it. Land treatment was the only natural treatment which could not be improved upon: with artificial treatment there was no finality, as with further investigations new methods would be suggested. Some might even think that the Pretoria methods had already been superseded by the activated-sludge process. Even if sufficient land could not have been found in the immediate neighbourhood, the works could have been located in the nearest open country. Land would always realize its value, but artificial sewage-purification works had no value when they had to be abandoned. He was pleased to note that under the guidance of Mr. G. Storrar, the Town Engineer, and Dr. J. J. Boyd, the Medical Officer of Health, the management of the Pretoria Sewage-Works was so efficient, as he was satisfied from long experience that the success of any works, good or indifferent, depended very largely on the skill shown by the management.

Mr. Temple.

Mr. F. C. Temple observed that South African and Indian experience appeared to agree as to the desirability of dealing with the sewage as fresh as possible: if the sewage was fresh, the sewers could be ventilated through house ventilating-stacks, rendering the master trap unnecessary. Recent experience in India also showed the advantages of making each part of a disposal-system as simple as possible, and of dividing the processes into separate tanks instead of attempting to deal with two processes in combined tanks. Revolving sprinklers operated by the flow of the sewage had not found much favour in India, for their action was interfered with by strong winds. It was difficult also, on account of caste rules, to find men to keep them in order. It was not easy to determine the average strength of the sewage at Pretoria, which appeared to be about 38 gallons per head in dry weather. That was weak compared with that of an up-country town in India. It was not clear whether the sewage of 10,000 persons, handled by the pail-removal system, to which 54,000 gallons of dilution water was added, was included in the 1½ million gallons. The strength of that part of the sewage was 5½ gallons per head, which was strong. The sedimentation-tanks had a capacity of one-sixth of the total daily volume, which was 240,000 cubic feet; thus the tank-capacity was 1 cubic foot per head of population served, which was just half the size of septic tank found most satisfactory in India. Presumably the use of separate septic tanks for the sludge made it possible to reduce the size of the sedimentation-tanks. The sixteen oxidizing-beds, each being 30,240 cubic feet in content, contained a total of nearly 484,000 cubic

Mr. Temple. feet of filtering medium. That was 12 cubic feet per user—four times the quantity allowed in many Indian installations. With so much filtering medium the high standard of the effluent was not surprising. The type of modified grit-chamber was interesting. It would be useful to know why the drop at the end of the chamber was so great, and whether so much head must necessarily be lost.

The Colombo drainage-system was surprising in its size and character to one who had worked for many years for eastern municipalities. For an Indian municipality to embark upon a sewerage scheme at all was rare. To construct a separate system was still more rare, for very few towns would face the cost. It would be interesting to know whether any fall of rain since the rain-water sewers were constructed had caused flooding. He had on many occasions discussed the Colombo disposal-works with Mr. M. R. Atkins, M. Inst. C.E. There was some unexplained difference in the behaviour of the septic tanks, as originally constructed, from that of some tanks in India. At first the tanks had a capacity of about 2 cubic feet per head of population served, which agreed with Indian practice. They were then admittedly overloaded, and produced much gas, the ebullition of which brought sludge to the surface. In India such overloading had produced in some tanks where the dilution was small (about 3 gallons per head), masses of scum consisting of almost unchanged fæces, with a highly-offensive and putrefactive effluent, and in others, where the dilution was great (about 40 gallons per head), practically no scum, and an effluent that was little else than an emulsion of crude sewage. In neither had there been an excessive quantity of gas. The entire difference of behaviour in the sewage was due perhaps to some difference in the design of the tanks. It would be interesting to know why 20 minutes' flow had been taken as the size of the detritus-tanks, and 12 hours' dry-weather flow as the capacity of the septic tanks, and why the particular size and depth had been chosen for the continuous-flow filters.

The sewerage system described by Mr. Lloyd-Davies was stated to be "partially separate," the sewers being designed to carry the rain-water falling on only 100 square feet per house. That being so, the necessity for such elaborate methods of calculating the size of the sewers was not clear. Whether the storm-water overflows came into action after the storm-water had entered the sewers, and acted by extracting it, or at the points of inlet to the sewers to prevent its entrance, was not stated. The latter alternative was much to be preferred, but the former appeared to have been employed. It would be interesting to know why six times the dry-weather flow had been selected as the capacity of the pumps, and one-quarter of

Mr. Temple. the average dry-weather flow as the capacity of the storm-water tanks. The reinforced double-storey tanks at the disposal-works were designed to deal with the whole of the daily dry-weather flow in 18 hours. No reason was given why that figure had been selected, and it was not stated whether it referred to the ultimate sewage of 290,000 persons or to the present population. The sewage was taken as being about 25 gallons per head. Indian experience tended to indicate that for a 25-gallon sewage a 12-hour period in the tanks was sufficient. In other words, for an average sewage the capacity of the tanks should be about 2 cubic feet per head, so that 25 gallons, or 4 cubic feet per head, would pass through in 12 hours. The depth of the percolating-beds did not appear to be given, so it was not possible to ascertain how much filtering-medium was provided.

Mr. Tickell. Mr. Tickell, in reply, stated that the omissions remarked upon by Mr. Braine were due to lack of space. The Paper on Colombo described a work of 20 years duration, and many interesting details had been necessarily omitted. The position of the Northern outfall had been selected so as to give the shortest length of main sewer. The reasons for abandoning the original site at the mouth of the Kelani were stated in the paragraph commencing "The Drainage Scheme." The cast-iron outfall pipes were only of short lengths and avoided heavy construction at the river-bank. The excavations in treacherous ground were carried out under very careful supervision by experienced foremen. Vertical runners, 9 inches by 2 inches by 12 to 15 feet long, were kept driven down a little below the excavation; 10-inch by 4-inch walings and struts were used, and cross polings and straw were put behind the uprights. In crossing the lake and standing water in the swamps an embankment was tipped ahead of the work, which acted as a coffer-dam on each side of the trench. Subsoil drains were laid in the bottom of the trenches, leading to sumps where pumps were kept going day and night. The coffer-dam in the sea at the Southern outfall was of substantial design with a framework of 12-inch by 12-inch timbers which formed a gantry for the steam pile-driver. The piles were 12 inches by 6 inches, close-driven, except at the deepest end, where steel interlocking piles were driven. The work was made sufficiently watertight to be pumped out, and the concrete bed and the cast-iron pipes were laid in the dry. With regard to the provision for discharge in the rain-water drains, experience had proved that the capacity of the drains actually built was sufficient. Considerable areas which were subject to flooding in rainy weather had now been reclaimed. It would have been manifestly extravagant to construct rain-water drains to carry off the record rainfall of 5 inches in an hour, Mr. Tickell. which might not recur for many years. During such a storm all the streets were flooded and water was everywhere flowing off the surface, and filling all drains and surface channels. The existing rain-water drains cleared the streets usually within an hour after a storm. The provision for rain-water in the sewers (six times the dry-weather flow) was an almost insignificant fraction of the rainfall, but it had so far proved to be somewhat in excess of the requirements; it might, however, be anticipated that, with the extension of house-connections, more rainwater would reach the sewers in future. The proper design for sanitary appliances for use by Orientals had been carefully considered, and a pamphlet on the subject prepared by Mr. Tickell had been published by the Ceylon Government to encourage their use in Colombo. The delay in making house-connections had been repeatedly brought before the Municipal Council, and it had been pointed out that the benefits of the expenditure were not being reaped in consequence. But the sanitary condition of every district had been immensely improved by the introduction of the public latrines connected with the sewers.

Mr. Lloyd-Davies. Mr. LLOYD-DAVIES, replying to the Discussion and Correspondence, stated, with reference to Professor Snape's remarks, that the sludge was allowed to remain in the lower chamber of the double-storey tank until complete digestion had been attained. The period required varied under different conditions, and seldom exceeded 4 months. It was not advisable to decant the sludge before putting the tank into operation again. The character of the sludge agreed with the description given by Mr. O'Shaughnessy: it was quite inodorous, very mobile, and did not attract flies. Referring to Mr. Midgley Taylor's and Mr. G. W. Humphreys's queries, the land on the Cape Flats would readily absorb the effluent from the purification-works, but it was not suitable, in Mr. Lloyd-Davies's opinion, for the treatment of crude sewage without nuisance, owing to the fineness of the sand. The major portion of the sewage from the unified area of Capetown was discharged into the sea at points outside the limits of Table Bay, where the prevailing conditions were fairly favourable. The conditions within the limits of Table Bay were unfavourable for the discharge of crude sewage, and there was no eligible site for a pumping-station and purification-works along its shores.

27 February, 1923.
WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The discussion on Main-Drainage and Sewage-Disposal **Works** was continued and concluded.

6 March, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Cuthbert Brown, M.B.E.
Robert Davison, B.E. (*Royal*).
Thomas Edward Hett Heywood.
Armell Richard Pollard, B.A. (*Cantab.*).
Clifford Dommett Sheldon, *D.S.O.*
Howard Lecky Sikes, B.A., B.E. (*Royal*).
Vincent Turner.
John Henry White, C.M.G.

And had admitted as

Students.

Reginald Ernest Adlington.
William Moffat Anderson, B.E. (*Adelaide*).
Edward Austin Blackwood.
Marix George Bucknall.
Michael Guy Callow.
David Hugh Cameron, B.Sc. (*Edin.*).
Percy Cave.
William Francis Cope, B.A. (*Cantab.*).
Edmund Stanley Dean, B.Sc. Tech. (*Manchester*).
Robert Laird Mackie Dick, B.Sc. (*St. Andrews*).
Sidney Edward French, B.Sc. (Eng.) (*Lond.*).
Frank Gibbons, B.A., B.A.I. (*Dubl.*).
James Paterson Gibson.
Charles Edward Hollinghurst.
John Alfred Golding Howard.
Thomas Trajan Lambe.
Arthur Turner Lancaster.
John Cameron MacGregor.
George William Moss.
Charles William Odling-Smee.
Arthur John Rusk, B.Sc. (*Edin.*).
Felix Henry Sharpe.
Harold Edwin George Stripp.
Raymond Faulknor Tremayne.
Camille Alfred Joseph Turlot.
Harry André Louis Vaugon, B.Sc. (Eng.) (*Lond.*).
Alexander Vellan, B.Eng. *Sheffield*).
George Walter Warr, Junr.

The Scrutineers reported that the following Candidates had been duly elected as

Members.

Henry Dacre Madden.
John Miller, LL.D., B.E. (*Royal*).

Associate Members.

Arnold Brooksbank.
Bhupalam Suryanarayana Chetti, M.A., B.Sc. (*Edin.*).
William John Connolley, B.Sc. (*McGill*).
Henry Merry Dunbar, B.Sc. (*Edin.*).
Harold Eccles, M.C.
Frederick Alfred Greaves, B.Sc. (Eng.) (*Lond.*).

***Associate Members*—continued.**

Arthur Illtyd Jenkins, B.Sc. (Eng.) (*Lond.*).
Alfred Illtyd Webber Jones, B.Sc. (Eng.) (*Lond.*).
Herbert George Kemp.
Alexander Paterson Laing, B.Sc. (*Edin.*).
Edward Arthur Lister, Stud. Inst. C.E.
John Vivian Lord.
John Murdoch Main, B.E. (*Sydney*).
Geoffrey Liddiard Newton, B.C.E. (*Melbourne*).
Anthony O'Farrell, B.E. (*National*).
Percy Ralph Robinson, B.Sc. (*Victoria*), Stud. Inst. C.E.
John Robert Sarolea, B Sc. (*Edin.*).
Edgar Walter Savours, Stud. Inst. C.E.
Hubert Leslie Torr.
James Paton Watson, Stud. Inst. C.E.

(*Paper No. 4411.*)

"Some Problems connected with the Rivers and the Canals in Southern India."

By Joseph Melville Lacey, M. Inst. C.E.

The watercourse of a large river of South India is, except during freshets, a comparatively narrow channel wandering in a vast bed of sand, which is fringed with banks of alluvial soil. The sand forming the river-bed is generally of great depth, and has little cohesion. The dry-weather channel of flow wanders usually in sinuous curves with a forward movement, the oscillations being as regular as the waves produced by shaking a rope held in the hand. Towards the end of the dry weather the watercourse consists of deep pools of practically still water, in which the speed of the current is hardly perceptible. These pools are connected by shallows, where the inclination of the bed and speed of the current are considerable. During floods this dry-weather channel becomes what may be termed the "live stream" of the river, moving at a considerably greater velocity than the rest of the water flowing down the river, and its sectional area is increased. The flood also produces a vast body of water moving at a less velocity than the live stream, stagnant in places, or even at times moving in an opposite direction to the usual current, especially where the river-bed widens out suddenly. This body of water forms the internal spills from the live stream, which flow across the sand-shoals into creeks; these, in their turn, become subsidiary channels. The spills eventually fall back into the main live stream. As the flood subsides the spills decrease until the live channel is capable of carrying the flood-waters. The width and depth of the live channel are dependent on the velocity of the flood flow in the channel and on the weight and grade of sand forming the river-bed. As the flood further subsides, the live channel diminishes in depth and width.

Typical features of rivers that scour deep channels between wide shoals of sand are :—

(1) Live channels which scour their beds in rising floods.
(2) Internal spills which cross the shoals obliquely and, under normal conditions, carry the surplus flow from the live channel to the creeks.
(3) External spills which carry the water over the river-margins, the latter being in some cases protected by levees.
(4) Live creeks which flow freely in floods without developing serious scour.

The sands deposited during falling floods form :—

(i) Spits, the result of side erosion at bends.
(ii) Bars left by live channels at the mouths of dead or dying channels.
  Dead channels, i.e., extinct bends cut off from active flow by a bar.
(iii) Shoals formed by deposition where a live channel bifurcates.
(iv) Glacis deposited beyond the edges of live channels by internal spills.

High sand-banks in sluggish water, particularly when the growth of long grass and shrubs is unchecked, may become in time permanent islands.

There is a natural tendency in any live channel which once attains to a rocky flank to adhere to that flank. In such cases the live channel may cease its sinuous course in the river-bed, and become "jammed" against that flank, resulting in most cases in inordinate scour coupled with reduced width.

The sand at the bottom of a live channel would seem to be in motion to a considerable depth, and all such channels have a natural depth of scour and a natural width of channel which depend on the velocity of the current and on the size and weight of the sand-grains of the river-bed. No assignable width of channel will induce the flood-water to flow through it with diminished depth. Apart from what is rolled along the bed, if the ratio of material carried in suspension to the volume of water is constant, and if the volume of water flowing down the live channel is constant, the greater the depth of the channel, the greater is the velocity required to prevent deposition of silt. If the velocity attained is unable to produce further scour, the sides are eroded, and the width of the channel is increased, When the limits of erosion of the bed and sides are reached, the channel spills. The natural width and depth of such a channel for any grade and weight of sand-grain, and for any given velocity, has yet to be determined.

It is obvious that measurement of the discharge of such a river, when in flood, is a difficult matter. The usual method of obtaining the hydraulic mean radius of a straight reach, observing the fall of the water-surface (if it were possible to do so with any degree of accuracy), and calculating the mean velocity of flow by known formulas, is inapplicable. "Bridge men" in India make the rough assumption that the live channel carries two-thirds of the flood discharge when fully scoured, and that the live creeks carry the remainder. Therefore, if the actual width of a well-defined single live channel of a river can be ascertained for a maximum flood, it should be possible to guide the whole discharge of the river into a single channel half as wide again as the single live channel, provided that there is no restriction to the free scouring of the bed of the channel to hinder the attainment of the natural maximum depth under a moderate afflux.

Before any scheme for the conservancy and control of a river is formulated, surveys of the bed of the river should be made for several dry seasons, extending some distance above and below the limits it is proposed to conserve. These should show the live channels, abandoned channels, and creeks. The configuration of the sand-shoals should also be delineated, so that it should be possible to ascertain the direction of flow of the spill water from the live channel to the creeks, and the consequent movements of the sand-shoals. A series of such surveys would show the changes taking place in the channels, and should give an indication of the action to be taken either by training-banks, or by dredging, in order to re-open abandoned channels, or by a combination of both to preserve the live channel in its regular curves, so as to prevent its attacking or hugging the river-margin. The sands forming the beds of the rivers of Southern India are generally derived from granitic formations, and the grains are heavier and larger than those of the sands forming the beds of the rivers of Northern India, which are derived from rocks of more recent formation; consequently the channels in the river-beds are more stable in the former than in the latter case.

Bank or Marginal Erosion.—The erosion of the margin of a river occurs chiefly at its concave bends, generally during falling floods. During a rising flood ground water in the river-margin is prevented from flowing towards the river by the high water-level existing there. Thus the hydraulic gradient of the ground-water plane of saturation slopes away from the river, and, while the river is in full flood, it tends to support the margin. When the level of the water in the river falls, the hydraulic gradient of the ground-water plane of saturation is reversed, and the water in the soil forming the river-margin, which is now supersaturated at its lower levels, flows outward into the river, carrying particles of soil with it. The soil, as it exudes on the face of the margin, is carried away by the current, hollows are formed, and, the support of the water being withdrawn, the margin caves in. This process continues until the velocity of the falling flood is unable to carry away the oozing soil, and a deposit is formed which checks the flow of the soil particles. Such erosion is worst and lasts longest when a considerable depth of water occurs against the bank, and when the current has sufficient velocity to transport the particles of soil; this combination is generally found at the concave banks of bends in the river. A further reason for the erosion of river-margins is that the banks of the lower courses of most large Indian rivers are composed of recent alluvial deposit; and the flood waters percolating into the already-saturated soil of the river-margin may form a semi-fluid pocket at its base. When the flood-level falls, the support of the water against the margin is withdrawn, and the margin may cave in. That the current has some erosive action is evidenced by the fact that, where a river traverses regions of alluvial deposit, strata of hard clay, or older alluvium, are met with, on which the current of a river in flood makes no visible impression, while the more recent alluvium is easily eroded. Further, in a rising flood the waters are fully charged with silt, while in a falling flood they are comparatively clean, hence their scouring action and transporting power are greater.

Any sudden depression or abrupt projection in the margins or even in the bed of a river produces a series of whirlpools and backwaters which may result in erosion. If the high floods of a tributary, entering the margin of a river obliquely, precede the flood in the main stream, erosion below the junction would be more rapid on the opposite bank of the main river, and this result may affect the relative cutting of both banks all the way down the main stream. When a bend becomes so sharp that the radius of the loop becomes less than the width of the river, the loop widens out. It has been stated that as a rule when a widening of the loop occurs, either due to this cause or to a projection in the river-bed or margin, the loop is not widened to such an extent as to make the versed sine of the arc greater than half the chord of the loop.

There is another force acting on the waters of a river which has not received due consideration, and, although allusion has occasionally been made to its existence, no definite pronouncement has been made as to its influence, particularly when the river is in flood, and when the flood water is moving with considerable velocity. This force is due to the rotation of the earth causing any moving body on the earth's surface to be deflected to the right in the northern hemisphere and to the left in the southern hemisphere. That such action takes place is apparent from the tendency of rivers in the northern hemisphere, especially those flowing in a north and south direction, to erode their right margins. Sometimes in extraordinary floods, where conditions and levels are favourable to rapid erosion, a river has taken a new course, and, other things being equal, the new course has generally been to the right of the old one. The Palar river, which now flows into the sea south of Madras, is stated by geologists to have flowed at one time into the sea some distance north of Madras; and the presence of the black alluvial soil known as Madras Clay is due to the fact that Madras once formed part of the delta of the Palar. The River Indus has a general tendency to erode its right margin, notwithstanding the attempts made to check such erosion. The live channel of the Godavery river above the railway-bridge at Rajahmundry (when the bridge was first under consideration in 1893) used to flow down what is known as the Rajahmundry channel, which was nearer the left margin than the right. The live channel has now merged into what was known as the Kovvur Creek on the right margin, and the channel hugs the bank for some considerable distance above the railway-bridge, causing erosion on the right margin and accretion at the left; the Rajahmundry channel has now become a creek. Just above the railway-bridge the live channel crosses almost at right angles to the direction of flow, from the right to the rocky margin on the left, on which the town of Rajahmundry is built, and where there is always a deep pool. The erosion of the right margin above the railway-bridge would have been considerably greater were it not for a stratum of dense hard black clay underlying the more-easily-eroded recent alluvium; but the danger of a break of the river at this place towards the low-lying Collair lake exists. The break of the Colorado river into the Salton Lake, the change of course of the Brahmaputra river from the Mayan river to the Jamuna river, the increasing size of the Goral river, which may yet form the main Ganges river, the tendency of the floods in the Orissa rivers to work over to the right, and of the floods of the Brisbane river to work over to the left, may be cited as examples of the deflecting-force arising from the earth's rotation.

Flood-embankments or Levees.—When a river brings down a vast quantity of material, either in suspension or rolled along its bed, which may consist of particles of clay, sand, shingle, or boulders, and when such a river spills its banks, its velocity is checked as it debouches from the hills into the plains, and the boulders, stones, and shingle are spread fan-wise over the country, the point of the fan being the point of debouchment. The river generally occupies a channel on the ridge of the fan, the valleys on either side of the fan being drained by smaller streams which discharge the spill-water of the river, and the local rainfall on their basins. Similarly, when a river approaches the sea, its velocity is checked, it spills over its margins, and the silt and alluvium also form a fan-shaped deposit, but this is not so pronounced as in the case of the boulder and shingle deposit. As the flood spreads over the great alluvial plains, the farther it travels from the river, the less material is deposited, consequently the largest deposits occur in close proximity to the river, which frequently occupies the highest ground. The level of the river-bed is gradually raised, and, if its banks are not protected by levees, the external spills cause a corresponding rise of the land on either side. If levees are constructed, the flood-waters are confined between the marginal flood-embankments, and the land adjacent to the margin is deprived of its layers of deposit. When these levees were constructed by early irrigation engineers, it was considered that, by holding up the flood-waters, the depth in the river would increase, and that corresponding scour and lowering of the river-bed would result. This, however, proved not to be the case. In many rivers the effect of the levees was to raise the river-bed so that the flood-waters topped the levees; and, as these were again raised, a corresponding rise of the river-bed took place, until in some places the river-bed rose above the level of the adjacent country. In an anonymous pamphlet published in 1858, addressed to "Red Tape" and signed "Delta," regarding the flood-embankments being constructed along the margins of the Orissa rivers, the following proposition was stated. DEK (*Fig. 1*) represents the longitudinal section of the bed of the deltaic portion of an inundating river between its delta-head A and the sea. A straight line DE is taken to represent the bed, instead of the curve which a river-bed takes owing to the gradual diminution of slope as it approaches the sea. If the volume of water entering the river at A is $Q + q$, and if the amount which the river is capable of carrying is Q, the volume which spills externally is q. If AD is the depth in the river at the delta-head corresponding with the full discharge of the river $Q + q$, and if BE is the depth in the river at B corresponding with the discharge Q, the flood line may be represented by the line

ABC. If the river is confined by longitudinal flood-banks, the whole flood $Q + q$ is passed down the river. If the tidal compartment of the river is of such a width that it is capable of discharging only Q, when charged with the whole volume $Q + q$ it will retard the flow in the upper or "fluvitable" compartment and produce a rise in the river to B′, giving an increased surface fall in the tidal compartment and a rise in the flood line of the river from B to B′. The effect of raising the flood line from AB to AB′ raises the river-bed to DHE, parallel to AB′, because the bed of a river always tends to become parallel to its flood line. If, on the other hand, the tidal compartment is more than able to carry the discharge $Q + q$, the result will be an increased discharge at B; this causes the bed of the tidal compartment to be deepened, and the slope to be increased until equilibrium is established. The writer of the pamphlet arrived at the following conclusions:—

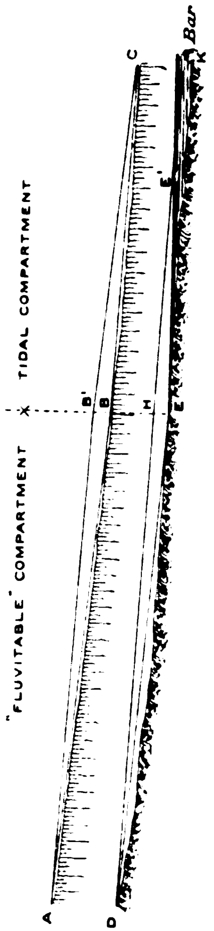


Fig. 1.

(1) If the tidal compartment is not capable of discharging the whole of the flood in a river, and if the flood is confined within the fluvitable compartment by marginal flood-embankments, the flood line and bed of the river will be raised, and the range of tidal influence will suffer diminution. The more the fluvitable compartment is worked beyond its capabilities, the more inefficient does it become owing to the rise of the river-bed.

(2) If the tidal compartment is more than able to carry off the discharge of the fluvitable compartment, it will cause the fluvitable compartment waters to scour and improve this compartment in the lower stages of the

> flood, and to increase the range of tidal influence. The more efficient the tidal compartment, the less will be the rise of the bed produced in the fluvitable compartment during high flood, and the better able will the tidal compartment be to reduce or to annul the rise of bed in the fluvitable compartment during the lower stages of the flood.

"Delta's" advice was not taken, and the result of embanking the Orissa rivers was that in 10 years their beds were reported to be perceptibly rising, as evidenced by the observed diminution of the range of tidal influence. The floods at the head of the delta had risen many feet above the highest ever known; and midway between the delta-head and the sea the embanked branches of the Mahanaddi were capable of discharging only half of the volume of a maximum flood.

More evidence of the effects of flood-embankments on the deltaic portion of a river seems necessary before any theory as to the result of their employment can be propounded. Speaking generally, they deprive the adjacent lands, not only of the fine rich deposit which would renovate and raise the soil, but also of the other benefits resulting from their inundation by gently-flowing waters. These form, in the course of time, natural drainage-channels for themselves, by which, after accomplishing their mission, they would pass off to the sea. On the other hand, by confining the flood-waters between embankments, the flood-level is raised, and, if a breach occurs, a roaring torrent of water is let loose, which deposits a coarse useless silt over the land, carries destruction in its path, and forms swamps in the lower portions of the adjacent country.

Silting-up of Canals.—In this connection, the Author refers not to the deposition of alluvium readily carried down in suspension, which renders the waters turbid, and which only settles in recesses and sheltered places beyond the influence of currents, but to silt composed of sands of various sizes, which are carried or rolled along the beds of channels, and which settle in proportion to their coarseness as the current slackens. It has been stated that a canal or channel which takes off from a river with a sandy bed must silt up rapidly if the velocity across the mouth of the canal exceeds the velocity in the canal. There seems to be some argument in support of this statement, as the velocity in the river across the mouth of the canal is sufficient to move forward the grade of sand forming this portion of the river-bed; and any diminution of velocity in the canal will result in a deposit of sand on the canal-bed until the surface of the sand in the canal has assumed a sufficient slope to give the water entering the canal the velocity necessary to move the sand along with it. It has also been stated that the ratio of bed-width to the depth in the canal should be equal to the ratio of the bed-width of the river to its normal depth. It is possible that the combination of the two conditions described may result in producing a non-silting canal.

The power of water to transport silt varies directly as its velocity and inversely as its depth. Water rolls rather than slides, hence only a certain quantity of matter can be held in suspension with a given velocity and depth. It may be accepted as generally true that there are three layers of solid matter in any open stream or canal. In the upper layer the current carries alluvium, or any light material that may be present. In the middle layer solid matter of higher specific gravity is suspended. In the lowest layer solid matter of still higher specific gravity is moved in suspension or rolled along. In most of the rivers of South India the silt or sand carried into the irrigation-canals is carried in suspension or, more generally, rolled along the bed in the lowest layer of the stream; and the depth of silt in the canal depends on the surface-level of the sand-shoal in the bed of the river adjacent to the canal-head. The bed of the canal at its head is generally at the same level as the mean bed of the river, so that in the low-water stages of the river the supply down the canal is small, In order to obtain the full benefit of the low-water supply, an anicut or low dam is built across the river, its height being such that the crest of the anicut is at the same level as the designed full-supply level of the water in the canal at its head. The effect of the anicut on the river is to cause the river-bed up-stream of the anicut to silt up almost to its crest, unless the velocity in the current is sufficient to scoop the sand over the anicut. The effect on the canal is to cause the canal-head to silt up almost to the same level as the surface of the sand in the bed of the river adjacent to the canal-head. The surface of the sand in the canal assumes a slope somewhat greater than that of the canal-bed fall, and ends abruptly in a slope corresponding generally with the angle of repose of the sand. In former years this silt was cleared at the close of each irrigation season, until, in 1892, Colonel A. W. Smart, R.E., pointed out that the silt at the canal-head was deposited with the first freshets that came down the river, and the only advantage gained by annual silt-clearance was a full discharge in the canal for a few days at the beginning of the irrigation season. He showed also that the level of the silt in the canal, which he termed the "normal silt-level," remained constant throughout the irrigation season. The conclusions to be drawn were that the annual silt-clearance was of little value, and that, if the supply passing down the canal was insufficient for the demand, raising the crest of the anicut so as to give an increased depth in the canal might result in a corresponding rise in the river-bed, thus the obvious course was to widen the head-reaches of the canal.

Most of the more important anicuts have scouring-sluices at their flanks in order to enable a deep channel to be scoured in front of the canal-head. These sluices, however, have only a local effect, and the next questions that arose were whether the sand in the river-bed could be maintained at its original level if the whole anicut was made in the form of a vented dam; and whether the bed of the river could be scoured and lowered and the canal kept free of silt, if the sills of the vents were placed at a lower level than the bed of the canal at its head. Several such vented dams have been built. These take the form of a bridge across the river. The vents are fitted with roller-bearing shutters, their span being 30–40 feet. The result is not as satisfactory as was anticipated, unless there is sufficient fall and velocity in the river to sweep the sand for some distance above the regulator through the vents. More extended observations seem necessary before any definite statement can be made that these regulators have solved the silt problem. Their cost is considerably more than that of the solid anicut, and the evidence in their favour is not sufficient to prove that the extra cost of such regulators warrants their construction.

It was also observed that some canals did not silt; and various theories were propounded to account for the circumstance. It was also noticed that some canals, which at one time silted badly, in course of time became free from silt, while in other cases canals which had been free from silt began to silt. As already pointed out, the silt or sand of the rivers of South India is carried in suspension close to, or is rolled along, the bed of the stream; and it seems possible that, if the canal-head takes off from a deep channel which may at the time be skirting the river-margin, and if the bed of the canal-head is above the level of the bed of the deep channel, no silt will enter the canal. As the deep channel of the river shifts in the course of time, an advancing shoal taking its place in front of the canal-head, the canal will silt up almost to the level of the surface of the shoal. Should the surface-level of the shoal be such that it is not covered with water during the low stages of the river, the head of the canal may be blocked entirely. The old Indian plan was to build the anicuts at an angle to the general direction of the river, the canal-head being situated opposite the down-stream flank of the anicut. The object of this arrangement was perhaps to force the current, which was obstructed by the anicut, towards the narrowing waterway between the anicut and the river-margin, and thus to scoop a pool in front of the canal-head. If an anicut were constructed curving diagonally down-stream and across a river, and if a powerful scouring sluice were placed at the down-stream flank of the anicut, it might be possible to maintain a deep channel in front of the canal-head. This suggestion requires consideration in the case of the small rivers of South India.

The Paper is accompanied by one diagram, from which the Figure in the text has been prepared.

(*Paper No. 4416.*)

"An Irrigation Project of the Californias."

By Sydney Lionel Rothery, Assoc. M. Inst. C.E.

THE PHYSIOGRAPHY OF THE AREA.

Situated north of the northern end of the Gulf of California, and astride the international boundary line between the United States and Mexico, being partly in the State of California of the former country, and partly in the State of Baja California of the latter, the present irrigated area is in that portion of the Colorado Desert which has been laid down in recent geological times, by the deposition of alluvium from the silt-laden waters of the Colorado river, in a low-lying basin forming topographically an extension of the great trough of the gulf. This river carries an enormous burden of solid material from the many cañons and basins of friable soil on its large watershed, which drains 242,000 square miles; and each year, during the passing of floods, a heavy load of silt is deposited over the delta regions in Baja California, behind long training-levees built and maintained, both for the protection from floods of the present irrigated lands and canal-system, and to direct the surplus discharge towards the gulf, against its present-day tendency to flow into the low-lying basin and to form a large inland sea.

Fig. 1, Plate 2, shows the present irrigable area and the main canal-system. The Salton Sea at the northern end of this area was formed by the inflow of the river into the basin in 1905–7. Its surface in November, 1920, was 249 feet below sea-level, and it now covers an area of 283 square miles, which is diminishing, chiefly due to evaporation, at the rate of 4 to 6 feet yearly, although it is the drainage-sump of waste irrigation waters. Its bottom has been given as 275 feet below mean sea-level.[1] The incursion of sea water from the gulf into such a low-lying area is prevented by a barrier across the south-eastern and southern edge of the irrigated area, which barrier has a lowest elevation of 32 feet above mean sea-level at Volcano

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[1] D. T. MacDougal and collaborators, "The Salton Sea: a Study of the Geography, the Geology, the Floristics, and the Ecology of a Desert Basin." (Published by the Carnegie Institution.) Washington, D.C., 1914.

Lake, about 45 miles north of the head of the gulf. The violent and high tides of the gulf, which have a maximum range of 37 feet, oppose the inflowing muddy waters of flood periods and have caused the backing up and wandering about of the river flow, permitting the deposition of silt and the more rapid formation of this barrier, thus cutting off the northern end of the trough of the gulf.

The deposition of the irrigable soils in the valley-bottom has taken place during those periods of time—sometimes amounting to several decades—when the barrier has obstructed the passage of the river to the gulf, and has deflected its flow into the basin. A change in the course of the river again to the gulf would permit the evaporation of the waters in the basin, leaving the deposited soils parched and arid in this almost rainless region until the next period of inundation. A diversion of the river into the valley would have taken place on various occasions since 1905, but for the operations of irrigation engineers. That such a geographic change is due is evident from the continued encroachment of the river against the levees protecting the valley lands.

THE PROJECT'S GROWTH, STATISTICS, AND JURISDICTION.

In 1900 no white men were resident in the valley, which then was desert land. To-day, between 50,000 and 60,000 persons are living there. Most of them are directly engaged in agricultural and farming pursuits, and all derive their livelihood as a result of the rapid growth of the project, whereby water in abundance has been diverted from the Colorado river and distributed over 940 square miles of dry soils. Railroads and a paved highway link the valley with the Pacific coast cities of Los Angeles and San Diego ; and direct train-services connect it with the eastern markets and cities of the United States.

The growth of the Californian portion of the project is shown by the following figures : In 1910 13,591 persons were registered in the valley, and the 1920 census returned 43,383. Products of 1910 were valued at $5,000,000, and those of 1920 at $65,000,000. This portion alone is the largest irrigation project in the United States, embracing 603,840 acres, of which 412,700 were irrigated in 1920. The diversion and delivery of water to this area is controlled by the Imperial Irrigation District, a municipal corporation having taxing powers, which is managed by a board of five elected directors. The water is delivered to the systems of thirteen mutual water companies, which in turn deliver and distribute it to consumers in the included area. The consumers are the owners of the water companies, the basis being approximately one share of stock per acre irrigated. The present purchase-prices of water stock vary for the different companies, being between $10 and $25 per share, a value that is added to the purchase-price of the land.

The principal field-crops are cotton, alfalfa, barley, milo-maize, canteloupes, wheat, and vegetables. The livestock comprises horses, mules, cattle, sheep, hogs, and poultry, valued at $4,752,000 in 1920. Fruits grown commercially are dates, grape fruit (pomelo), apricots, lemons, almonds, and grapes. Butter, milk, and honey are also produced in bulk, the value of butter made in 1920 being $3,377,000. Cotton and its by-products were valued at $23,957,000, canteloupes at $9,208,000, and grain at $5,007,040 for the same year. The yield of alfalfa is astonishing, averaging 6 to 12 tons per acre from as many as nine mowings per year. The purchase-prices of land are: Alfalfa land, $150 to $300 per acre; cotton or corn land, $125 to $200; and bearing vineyards and fruit orchards, $500 to $1,000 per acre. Rental values are between $20 and $60 per acre for agricultural lands, and higher values are obtained for vineyard and orchard land.

The revenue of the Imperial Irrigation District is derived from a levy on the assessed value of all irrigable property within its boundary, and from a charge for each acre-foot of water delivered to the water companies, by virtue of the authority vested in it by the California Irrigation District Act. The assessment rates fixed for 1920, 1921, and 1922 have been $1·90, $2·50, and $1·25 respectively, and the charge per acre-foot of water is $0·85. In 1920 1,218,096 acre-feet were delivered to the water companies. To finance the cost of improvements and extraordinary expenditures, funds are acquired through the sale of bonds from time to time as provided for in the Act; three bond issues since 1914 have made the total bonded indebtedness of the district $8,500,000. The water companies finance their expenditures for the maintenance, construction, and operation of their distribution-system by a direct charge for each acre-foot of water delivered to the users or stockholders, as well as from the sale of water stock.

The other portion of the project (on Mexican soil) has an irrigable area from the present canal-system of about 259,000 acres; and more than 140,000 acres were irrigated in 1920, using 572,430 acre-feet of water. The corporation nominally in control of the conveyance of the water through this area, and of the distribution to consumers, is the *Compania de Terrenos y Aguas de la Baja California* (*Sociedad Anonima*). By amendment to a law of the State of California in 1914, the Imperial Irrigation District was permitted to own the Mexican portion of the system, and the "Company of Land and

Water of Lower California" is virtually the Imperial Irrigation District operating in Mexico, necessarily complying with Mexican statutes, and deriving revenue from the tariff-rates fixed by the Mexican Government for charges for water. One of these statutes entitles these lands to 50 per cent. of all of the water conveyed through them, should it be wanted; 31 per cent. was the ratio delivered in 1920.

The fixed rate of $1·50 per 1,000 cubic metres (or $1·70 where the Compania builds the consumers' gates), is equivalent to a revenue of 87 cents per acre-foot, after deducting a tax levied on the allowable rate. The American consumer pays a fixed charge, plus an assessment on his holding, thereby contributing two to three times the price for the same quantity of water. The injustice of this, and the fact that the major portion of the revenue derived from the whole project must be expended in Mexico for the conveyance of the water through it, and for the assurance of adequate measures on the delta for protection from floods, have naturally caused agitation on the American side that the landed interests benefiting on Mexican soil should contribute a more equitable proportion of the expenditure.

FLOOD-PREVENTION PROBLEMS.

Although none of the irrigated land is contiguous to the river-channel, a study of the characteristics of the Lower Colorado river, and of the problems affecting control in the passing of the unused flow on its course to the gulf, is inseparable from a consideration of the welfare of the project.[1] Since 1900 the maximum recorded discharge of the Colorado at Yuma was 220,000 cubic feet per second, on the 22nd January, 1916, and the minimum was 2,700 cubic feet per second, on the 18th January, 1913. The greatest yearly volume of discharge was 25,967,577 acre-feet in 1909, and the least yearly volume was 7,959,000 acre-feet in 1902. The volume of discharge for 1920 was 21,446,691 acre-feet, in which year flood-water conditions reached a maximum, due to abnormal winter rains in Arizona on the Gila watershed, and to the late cold temperatures with snow-precipitation during April in Colorado and Utah. There is a flood each June caused by the discharge from the melting snows in the northern states, and in some years there occurs also a high flood of short duration (3 to 6 days) in January or February, due to rains in Arizona. *Fig.* 2 shows the fluctuations of discharge for the period 1912 to 1921. The duration of the annual summer flood,

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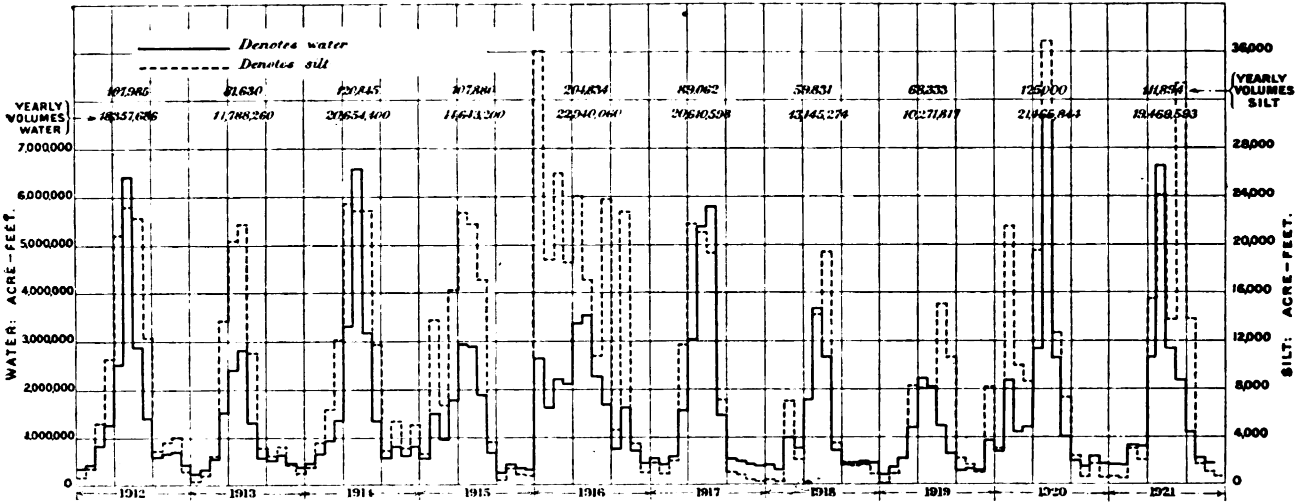
[1] Transactions of the American Society of Civil Engineers, vol. lxxvi (1913) p. 1201, and vol. lxxxi (1917), p. 297.

Fig. 2.



MONTHLY VOLUMES IN ACRE-FEET: COLORADO RIVER DISCHARGE AT YUMA.

i.e., when the flow exceeds 50,000 cubic feet per second, is from 3 weeks to 2 months; and during this period there is the potential danger of a change of course of the whole river into the valley lands. The channel to-day on the south-east and south of the irrigated area in Mexico is on the top of the delta barrier. Fig. 3, Plate 2, which is a profile from the gulf across the barrier at Volcano Lake to the Salton Sea, shows the ready fall towards the Salton Sea, 250 feet below sea-level, compared with the slight fall towards the gulf.

A winter flood of 1904-5, and the following summer floods of 1905-6, did actually break into the valley. The break occurred at A, Fig. 1. The task of closing the breach seemed almost hopeless, and it was not until February, 1907, that the final closure was effected, after much damage had been done to some of the pioneer land-settlements, entailing the ultimate bankruptcy of the corporation then controlling the irrigation-system. The great inflow caused the cutting of the Alamo and New river *arroyos* into deep precipitous-sided chasms for almost their whole length, thereby displacing by recession and erosion a volume estimated at 450 million cubic yards of soil, equivalent to almost twice the excavation made on the construction of the Panama canal, which volume was carried in suspension and deposited in the Salton Sea. The recession began at the Salton Sea end of the *arroyos*, and worked back with such rapidity that engineers feared it would reach the Colorado river before the closure was effected. Fig. 4, Plate 2, shows typical present-day cross sections of the New River channel, compared with others taken prior to 1905 at about the same locations, between the international boundary line and the Salton Sea.

In 1909 the river abandoned its southerly course to the gulf for a westerly direction into Volcano Lake (B, Fig. 1). Again, in 1918 and in 1921 there were marked changes in the direction of flow (C and D, Fig. 1), which show the tendency of the river to drop from the barrier on the north side and to flow into the valley lands. To oppose this tendency, 60 miles of training-levees have been built along the whole length of the barrier; these are known as the C. D., the Ockerson, the Saiz, and the Volcano Lake levees. The gradual raising of the river-bed and of the deltaic ridge behind the levee-system due to silt-deposition, and the wandering of the channel as a consequence of the reduction of bed-gradients, have been so rapid in the past 10 years that the last-named levee has been raised 10½ feet for a length of 6 miles on its western extremity to keep in advance of the self-elevation of the river, which amounted to 13½ feet in the same period. The

Ockerson and Saiz levees have also had to be raised several feet, and many breaches have occurred in all the levees, in spite of careful maintenance and the vigilant patrols during flood periods. Flattened slopes, dense and vigorous vegetation, and the high gulf tides restrict the discharge of the river at such times.

As evidence of the prolific growth on the soils in the delta subject to inundation, an area exceeding 40 acres was cleared flush with the ground, from the south bank of the river, along the site for a diversion-channel, in February, 1921. In September an impenetrable growth of young willows (*Salix spp.*) covered a large portion of the area to a height of 15 feet, and this was so thick that a line had to be brushed before men could walk through it. Arrow-weed (*Pluchea sericea*) also grows luxuriantly 6–10 feet high in impenetrable masses, and forms the brush of which mattresses are made. Cottonwoods (*Populus spp.*) and mesquite (*Prosopsis juliflora*) are the trees of larger growth which thrive in the region. The tule (*Typha latifolia*) grows in masses in swamps and lagoons similarly to the well-known bulrush.

Fig. 5.

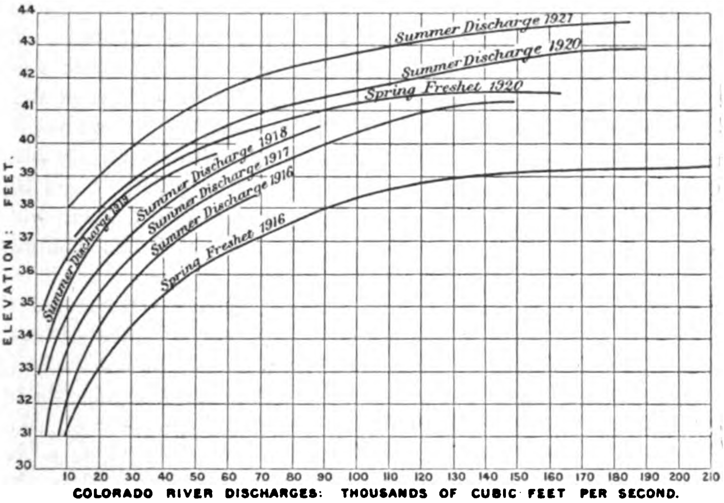


Fig. 5 illustrates how the flood waters have been increasing in height against the Volcano Lake levee since 1916; and the cross sections (Figs. 6, Plate 2) are proof of the rapid rate of upbuilding of the river-bed and banks in the short period of 2 years, during which but one flood occurred of sufficient duration to permit much silt-deposition on the river-banks. These and other similar cross sections were taken several miles apart on the westerly course of the river above Volcano Lake, and therefore show that the deposition was general. In this same flood fourteen wagons of a contractor's equipment were so completely buried that their platform tops, 4 feet above the bottom of the wheels, were just visible in the new general level surface of the ground; and excavation had to be made to recover the equipment in September, 3 months after it was submerged, at about ½ mile distant from the river.

Indications of volcanic activity exist on the western side of Volcano Lake, where there are solfataras, or mud volcanoes—hot springs emitting steam and sulphurous gases, occasionally erupting as geysers—and the inactive crater of Black Butte, which is a detached mound of burnt earth and scoriæ rising abruptly to a height of about 1,000 feet, close to the west end of the levee. The whole region is also subject to earth tremors. Geologists state that it lies on a fault plane which passes through the great interior valley of California and southward through the trough of the gulf. The mud volcanoes and hot springs are said to be due to the infiltration of water from the Colorado river overflow down on to the heated beds of rock not far beneath, where, becoming converted into steam which bursts upward through the deposits of silt, it forms encircling walls of mud around their orifices.

Consideration of all these facts—the continued tendency of the river to flow north, the obstruction to its flow towards the south, the gradual increase in elevation of the barrier, the consequent flattening of the river-gradients, which are now 0·85 to 1·25 foot per mile, and volcanic activity against the levee, on the north side of which exist steep gradients on a most friable soil surface where a river can entrench itself in deep channels by rapid recession—shows how important are the problems of protective measures, which are imperative for the very existence of the project. Further heightening and thickening of the levee-system can only be done at prohibitive costs, and such a remedy would not remove the potential danger of inundation of the valley during flood seasons, or terminate the necessity for still further raising of the levees each year as the elevation of the deltaic cone increases.

Investigations have shown that it is possible to divert the river through the south bank cone on to lower ground towards the gulf-head; and, when such a work is accomplished, the result should be the drying up of Volcano Lake and the lowering of the river several feet deeper into the ground up-stream from the diversion-point, due to recession and bed-scour, on account of the steeper gradient along the diverted course. This would give relief for a number of years, until the river again should have elevated itself on its new course to its present bed-elevation against the upper levee-system; but, before this condition again comes about, it is hoped that permanent relief from floods will have been obtained by the construction of a great storage-dam across the Colorado river in its lower watershed at Boulder cañon, which is to have a height exceeding 500 feet and a storage-capacity of about 25,000,000 acre-feet.

Work for the excavation of two channels through the south bank cone (X on Fig. 1), and for the obstruction of the river-flow just below them, is being (1921) vigorously prosecuted. Three 1½-cubic-yard dragline dredges and a 1-cubic-yard machine have been working day and night to construct stream-beds, on a gradient subject to erosion, of a combined capacity of about 10,000 cubic feet per second. It is hoped that this discharge will have sufficient erosive and transporting power to have merged both channels into one expanded channel large enough to carry 50 to 100 per cent. of the maximum flow by the time the June flood of 1922 will have passed. The diversion may not be completely efficient before the following year, for it is impossible to foresee or control all the vagaries of flood-conditions of such a wandering river.[1]

THE PROTECTIVE LEVEES.

Cross sections of the levees are shown in *Figs. 7, 8,* and *9*. The soils of which they are made are the fine alluvial silts and sands. These are excellent materials for working and compacting into dense embankments, but, being devoid of any true clays, they lack cohesion, powder up when dry under ordinary traffic, and offer no resistance to erosion when subjected to stream-action. Infiltrating water that acquires any velocity through the existence of a void in the fill, or through a ground crack below it, rapidly scours an opening by carrying the fine soils away in suspension. The patrol during floods takes prompt action to reinforce any apparent wet places with sand-bags or brush riprap, or to hold to as narrow limits as possible any breach that occurs and cannot be closed while the waters are up.

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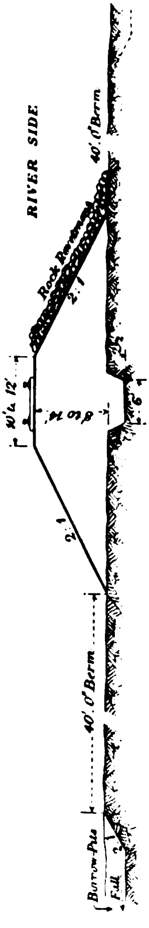
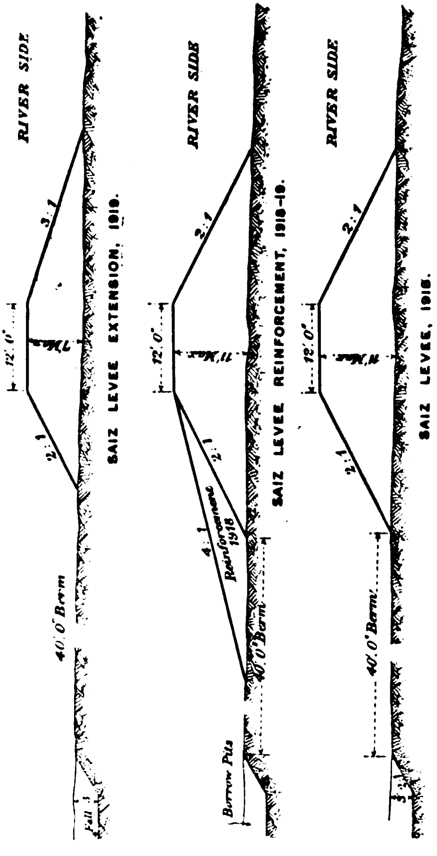
[1] Since this Paper was submitted the diversion has been successfully accomplished, the whole of the June flood of 1922 going the diverted route through the south bank cone. 657,000 cubic-yards of soil was moved by draglines, for the initial stream diversion, and the flood moved, by erosion and transportation, 15 to 20 million cubic yards within the length of dredging operations. (S. L. Rothery, "A River Diversion on the Delta of the Colorado," Proc. Am. Soc. C.E, vol. xlix (1923), p. 671.)—S. L. R.

Fig. 7.


RIVER SIDE
Rock Revetment
2:1
40' 0" Berm
20' to 12'
8' to 14'
6'
2:1
40' 0" Berm
Borrow Pits
Fall
2:1

Ockerson Levee.

Figs. 8.

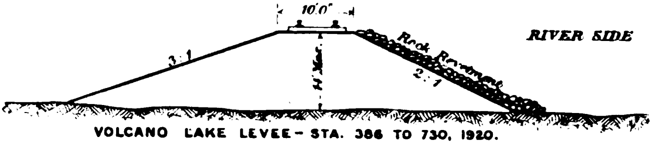

RIVER SIDE
3:1
12' 0"
14'
2:1
40' 0" Berm
Fall 1
RIVER SIDE
2:1
12' 0"
11'
2:1
2:1
4:1
Reinforcement 1918
40' 0" Berm
Borrow Pits
RIVER SIDE
2:1
12' 0"
11'
2:1
40' 0" Berm
3:1

SAIZ LEVEE EXTENSION, 1919.

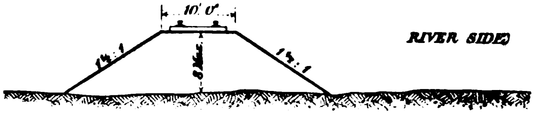
SAIZ LEVEE REINFORCEMENT, 1918-19.

SAIZ LEVEE, 1918.

Figs. 9.


10' 0"
3:1
14' Max.
Rock Revetment
2:1
RIVER SIDE

VOLCANO LAKE LEVEE – STA. 386 TO 730, 1920.


10' 0"
1½:1
8' Max.
1½:1
RIVER SIDE

VOLCANO LAKE LEVEE – STA. 98 TO 386, 1920.

Under the cross section shown for the Ockerson levee (*Fig. 7*) is an excavated trench which has been refilled with clean soil, extending along under the base of the levee. The function of this ditch is to close ground cracks which are formed in a stratum of soil composed of very fine loamy silt known as "adobe." Damp adobe is greasy to the touch, and a kneaded lump of it can be pared with a knife like soap, but it is not sticky like clay, and it will dissolve in moving water; on drying, it shrinks, forming wide cracks extending to 2 or 3 feet deep. The covering up of these cracks by wind-blown sand and vegetation accounts for their existence under the surface.

A diversity of opinion exists as to which is the better side of the levee to excavate borrow-pits. Sometimes they are made on both sides, being staggered, so that a cross section continued beyond the levee-slopes will only show one pit. All debris, roots, vegetation, and unbroken clods are removed from the soil, put into the levees, and the ground-surface under the base is thoroughly cleaned and ploughed to give the new levee a bond with the soil. During construction, teams are driven over the soil, which is dumped in layers of about 1 foot in thickness to assure a dense fill.

Burrowing animals, such as gophers and muskrats, dig their holes into the levee-sides. The flat slopes, however, provide a base of sufficient width to allow for their depredations, though this is not the case in the smaller canal levees, where, because of the permanent water-supply, such rodents are more numerous and give much trouble.

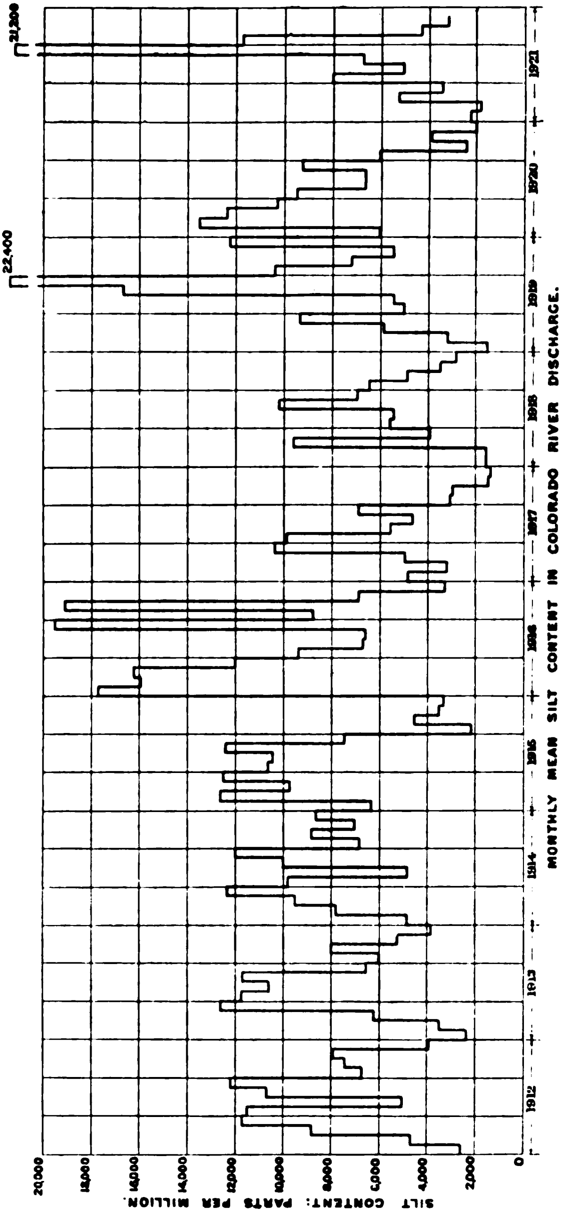
The side slopes on the river side of the C. D., the Ockerson, and the Volcano Lake levees, are heavily revetted with quarried rock, 2-4 feet thick, to provide a surface which resists the action of moving waters, and to protect the soluble soil in the levee from erosive action. Flat side slopes, short stub levees or buttresses at intervals along the main levee, wide berms between the toe and borrow-pits, transverse berms left between consecutive borrow-pits, and encouraging the growth of vegetation against the levee-sides, are measures that contribute to the stilling of the waters against the side slopes by deflecting the current to a safe distance from the levee. On the top of these rock-faced levees are railroad-tracks from the quarries, of which there are two, one at Andrade at the point of diversion for the canal-system, and the other on the east flank of the Cocopah mountain range, 4 miles west of Black Butte. The quarried rock, which is of granitic texture, is loosened in a vertical plane along the quarry-face by simultaneous blasting-charges, breaking 50,000 to 80,000 cubic yards into sizes that can be handled by 2½-cubic-yard Bucyrus steam shovels, which load it on to 16-cubic-yard side-dumping railroad-cars; it is then hauled to the levees, dumped, and spread over the side slopes with a Jordan spreader, at a total cost of 40 to 85 cents per cubic yard, including all quarrying charges.

THE RIVER'S CAPACITY FOR SILT-TRANSPORTATION.

The erosion, transportation, and deposition of soil are well-known features of the Colorado river. The variable discharge alternately scours and refills the channel-bed; the high velocity of flood periods carries along all the heavier sand and bed-silt, which is again deposited, miles downstream, as the floods subside. At Yuma the scour depth at high floods is about 20 feet lower than the usual bed of the river for low discharges.

This annual scouring (and in some years it takes place more often, there being two high floods and additional freshets) of several hundred miles of river-bed transports a great volume of soil. Assuming a conservative scour depth of 10 feet for 800 feet wide, the quantity of material thus transported per 100-mile length is nearly 100,000 acre-feet. Much of this probably represents the movement, by rolling along the river-bottom, of the heavier or coarser particles, and therefore may not be included in the estimated movement of silt, deduced from measurements of silt quantity carried in suspension; but as these same heavier particles probably travel more slowly than the flood wave, and are deposited again in the lower reaches of the river, it may be correct that they should not be included, though the transportation of such coarser particles to the sea or delta region is effected just the same, extending probably over a period requiring several floods to move them to their destination.

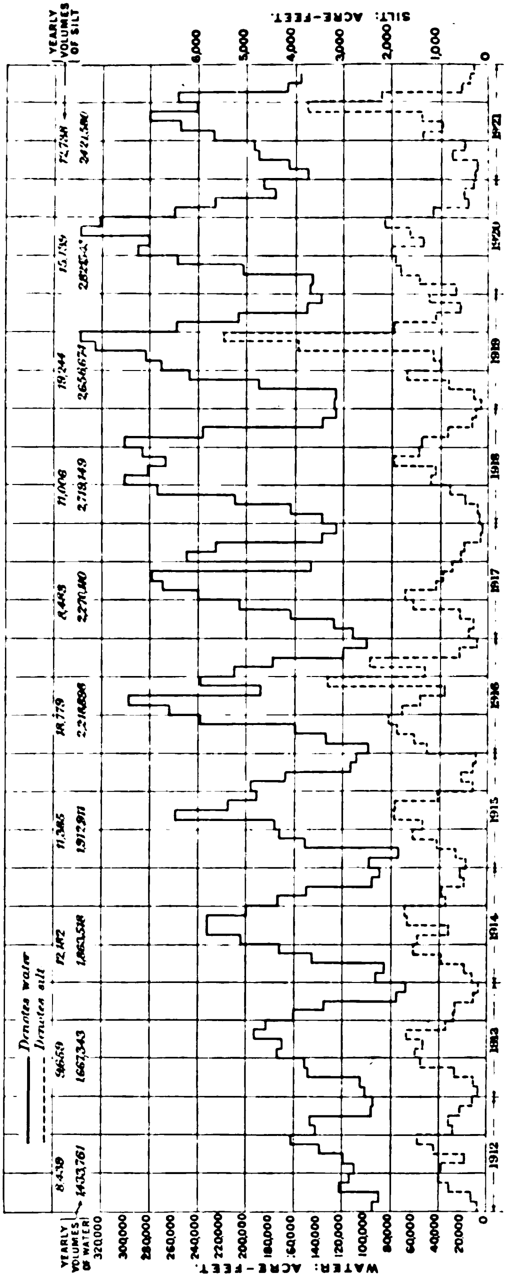
The silt curve of *Fig. 2*, p. 165, which has been prepared from measurements of the turbidity that have been regularly taken by the United States Reclamation Service, shows the estimated yearly volumes that have passed Yuma since 1912, and the monthly fluctuations of the silt quantity. The monthly values of the silt content, shown in *Fig. 10*, are averages of many samples taken several times each month. It is seen that the average silt content varies widely in different years, and it is not proportional to the amount of discharge, as will be apparent by comparing the curves. The annual content of suspended matter is stated by Mr. E. C. La Rue to average 7,000 parts per million and to vary between 2,400 and 12,000 parts per million. *Fig. 10* shows a range between 1,500 parts and 22,400 parts per million, there being 5 months in 1916, 1 in 1919, and 1 in 1921, in which the measure of turbidity exceeded 13,000 parts per million; but, as 6 months are only 6·6 per cent. of the whole period shown on the graph, they may be considered exceptional, and the extreme figures as given by Mr. La Rue limit the general variation. The specific gravity of the silt is given as 2·65, which corresponds with a weight of 165 lbs. per cubic foot.



MONTHLY MEAN SILT CONTENT IN COLORADO RIVER DISCHARGE.

Fig. 10.

Fig. 11.



Denotes water
Denotes silt
YEARLY VOLUMES OF SILT
YEARLY VOLUMES OF WATER
8,439
1,433,761
9,659
1,667,343
12,182
1,863,528
11,385
1,912,911
18,779
2,218,898
8,485
2,270,140
11,006
2,718,149
19,244
2,658,673
15,139
2,825,52
12,798
2,421,580
WATER: ACRE-FEET.
320,000
300,000
280,000
260,000
240,000
220,000
200,000
180,000
160,000
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
SILT: ACRE-FEET.
6,000
5,000
4,000
3,000
2,000
1,000
0
1912
1913
1914
1915
1916
1917
1918
1919
1920
1921
MONTHLY VOLUMES IN ACRE-FEET DIVERTED INTO THE CANAL-SYSTEM.

Fig. 11 shows the estimated acre-feet of silt per month for each year since 1912, which has been carried in suspension into the main canal and distributary system of the project. Some of it lodges in the canals and laterals, necessitating much dredging and cleaning, and some of it is spread over the irrigated lands, where it is claimed to have a fertilizing value due to the potash, phosphoric acid, and nitrogen content (1·21 per cent. of potash, 0·17 per cent. of phosphoric acid, and 0·83 per cent. of nitrogen are average values from an analysis of the river sediment made by Mr. R. H. Forbes).[1]

The acre-feet of silt shown in *Figs. 2* and *11* were computed from the formula :—

Volume of silt in acre-feet = quantity in acre-feet × mean silt ratio × 0·727, the mean silt ratio being the ratio of silt to water, that is, the number of parts per million divided by a million. The constant 0·727 is used to convert weight to volume—the mean silt ratio being determined by weighing the silt-residue after the evaporation of the water in the samples taken—and it is derived thus:

$$\frac{\text{1 cubic foot of water}}{\text{1 cubic foot of loosely-deposited silt}} = \frac{62\cdot 4 \text{ lbs.}}{86 \text{ lbs.}} = 0\cdot 727.$$

The Author understands that it was determined from a number of experiments, that the average weight of a cubic foot of silt, loosely deposited by water, was 86 lbs. after being dried. The weight-ratio of a cubic foot of water to a cubic foot of soil is thus 0·727.

On this estimate, the quantity of silt diverted into the canal-system of the project, in the flow of irrigation water, are given in the following Table :—

SILT QUANTITY.

Year.	Acre-Feet.	Cubic Yards.	Tons of 2,000 lbs.
1912	8,439	13,614,639	30,327,234
1913	9,659	15,582,865	34,711,548
1914	12,182	19,653,220	43,778,453
1915	11,385	18,367,420	40,914,225
1916	18,779	30,296,161	67,486,092
1917	8,482	13,684,010	30,481,763
1918	11,006	17,755,980	39,552,267
1919	19,244	31,046,345	69,157,163
1920	15,139	24,423,749	54,405,024
1921	12,738	20,546,394	45,767,093

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[1] University of Arizona, Agricultural Experiment Station, Bulletin 44.

The silt quantity in the water diverted for 1920, if evenly spread over the whole irrigated area for that year, would represent about ¼ inch depth ; its volume is equal to that of a cube of more than 870 feet on a side. The silt transported by the whole river flow (*Fig. 2*) was about 8¼ times this quantity. Mr. A. P. Davis, Director of the United States Reclamation Service, is the authority for the statement that the annual transportation of silt by the Colorado river averages 113,000 acre-feet, which is equivalent to a cube of 1,700 feet on a side.

It is interesting to compare the capacity of the Colorado river for silt-transportation with that of the Ganges. Assuming the Colorado summer flood to average a discharge of 100,000 cubic feet per second for 1 month (or one-twentieth part of the Ganges flood), it would in that time transport 1,944 million cubic feet of solids if its waters carried solids in suspension amounting, as an average, to 0·0075 of its bulk. The transportation of solids in the Ganges is given by Sir Francis Spring[1] as 13,000 million cubic feet, with an average of 0·0025. Thus the Colorado, with one-twentieth the discharge of the Ganges, transports one-seventh the total quantity of solids moved by the latter, its transporting-capacity being, therefore, nearly three times that of the Ganges. This comparison will serve to emphasize the importance of the silt question that has to be contended with in the irrigation project under consideration.

THE DIVERSION-WORKS.

The intake to the canal-system is in the extreme south-east corner of California, and is therefore on United States soil. Rockwood gate—the intake structure—is located in the river-bank, where the vertical range of the river's surface would be 22 feet, were it not that low-water conditions necessitate the construction each year of a temporary weir across the river. This conserves sufficient entry head through the gate to ensure a full supply for the offtake canal. The effect of the weir reduces the vertical range to about 14 feet.

A permanent dam or weir has never been feasible, because during high floods the scour depth of the river-bed is 20 feet below its normal level; and the townspeople of Yuma, in common with the surrounding rural population, vigorously oppose the placing of any obstruction to the free passage of floods that may cause inundation of their lands. The construction of a permanent sill at such a depth, with collapsible or removable gates, would be very costly and extremely difficult with such changing depths of the river-bed.

The weir now placed each midsummer is of the novel type shown in Figs. 12, Plate 2. A cableway spans the river, and anchor-pile clusters are driven 100 feet apart across the bed, and 120 to 150 feet up-stream from the cableway. Wedge-shaped brush mattresses, weighing about 7 tons each, are built on the bank, and to a point on the inclined surface of each is attached a wire rope 120–150 feet long, of which the free end has a large prepared loop.

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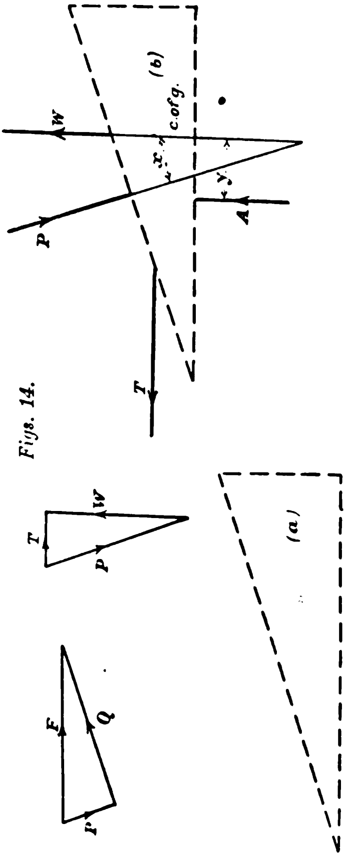
[1] Minutes of Proceedings Inst. C.E., vol. ccv, p. 87.

When the yearly flood wave has passed, and the flow has become less than 50,000 cubic feet per second, the placing of the weir units in the river is only a matter of hours for a crew of 10 men. A locomotive crane deposits each mattress under the cableway on the bank. A sling is passed around the mattress, and the traveller, operated by a 300-HP. electrically-driven hoisting-winch, carries the mattress out to its appointed place in the river and lowers it to within a foot of the surface. The loop is taken up-stream and passed over one of the clusters of anchor piles, care being taken to see that the loop goes down to the river-bed. The mattress is then released by operating the trip hook provided for holding the sling, and it dives to the river-bottom, against which it is held by the force of the current.

The units are placed side by side across the river, which here is 950 feet wide; and four to six mattresses are anchored to each pile cluster. The obstruction thus made across the stream causes silt-deposition up-stream and through the interstices of the brush in the mattresses, giving an efficient and economical brush and hydraulic fill dam.



Figs. 14.

FORCE DIAGRAMS FOR SUBMERGED MATTRESS.

Figs. 13, Plate 2 show the construction of the brush mattresses, and *Fig. 14* is a diagram of the forces acting on the submerged wedge. If F be the force of the flowing stream against the inclined surface

(*Fig. 14a*), Q and P are the components of F, parallel and normal to that surface. T denotes the tension in the wire rope, and W the force through the centre of gravity of the wedge; this equals the weight of the wedge minus the weight of displaced water. Since the three forces, P, W, and T do not meet in a point (*Fig. 14b*), they do not produce stability, which is due to a fourth force A, an upward pressure, so distributed and located that the sum of the moments about any point is zero. The tension T may be considered as effective through the centre of gravity of the wedge, when the moment Px is balanced by the moment Ay to produce equilibrium. Px is the moment that holds the point of the mattress down, and is the cause of the apparent diving action on one being submerged. This method of placing the mattresses is shown in the longitudinal section of the river, Fig. 15, Plate 2.

Rockwood gate, shown in Figs. 16, Plate 2, is 621 feet long between the end walls. The flow into the canal is regulated by seventy-five openings each 6 feet 6 inches wide, fitted with flashboards. By the consequent overpour discharge for the variable stages of the river only the upper portion of the water is taken, and much of the heavier silt and sand that travels along the river-bottom is prevented from entering the canal-system. The sill of one-half of the structure is 28 feet below the maximum high water, and that of the other half, 20 feet. A floating boom is placed parallel to the structure, 80 feet distant in the river, to deflect floating driftwood and trees. The openings are operated in groups and combinations of groups to produce the desired river-bed conditions at the approach to the structure. In this way the formation of sand-bars can be prevented, and channel-characteristics can be maintained, especially in periods of small river-discharge.

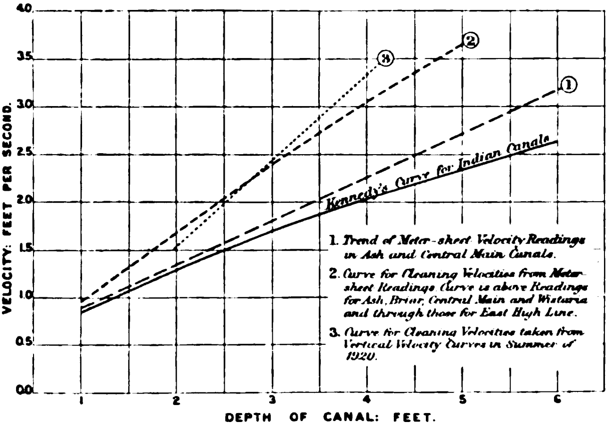
Across the canal, 6,000 feet below the intake structure, Hanlon gate is situated. This concrete structure was the former intake-regulator, and it contains a power-operated Stoney gate 25 feet wide, and nine undershot radial gates for submerged openings each 12 feet wide. Between the two structures, one 20-inch, and sometimes an additional 18-inch electrically-driven suction dredge, operate to remove bed-silt and sand that pass through Rockwood gate.

THE CANAL-SYSTEM.

The main canal follows the route of an old stream-bed, which has been transformed into a canal along its 48-mile length as a result of much dredging, channel-training, and elimination of bends. It has a capacity of 6,000 cubic feet per second. The main canals, and the two drainage-channels formed by the floods of 1905–6, are shown in Fig. 1, Plate 2. The whole of the supply for the United States portion of the project has to pass through some of the 130 miles of main canals on Mexican soil.

Higher velocities than usual are needed to ensure self-maintaining canal sections, on account of the large amount of fine silt carried in suspension. Mr. R. G. Kennedy's formula for critical velocities, used in India, $V = 0 \cdot 84\ D^{0 \cdot 64}$, would give velocities too low to prevent silting with such turbid water; and the coefficient should be increased to 0·98 to be applicable, making the formula $V = 0 \cdot 98\ D^{0 \cdot 64}$. *Fig. 17* shows the curve of Kennedy's formula compared with curves plotted from observations made on some of the self-maintaining canals and on those that require but occasional dredging.

Fig. 17.



Unfortunately, serious study was not given to the subject of ensuring critical velocities in the flows, prior to the construction of the canals: otherwise, much of the dredging now needed all over the project would not be required.

The hot climate is conducive to aquatic growth in the canals and laterals, of which the tule is most troublesome. Ploughing, scraping, and dredging are employed to remove the growth; the last method is preferable as the roots are taken out and the canal is enlarged and deepened at the same time. It is impracticable to keep the canal-banks free from growth at all times. The growth of grass at the edges of the water collects silt, and side slopes given to newly-constructed canals soon become bermed, so that the worn cross section usually becomes rectangular, with vertical edges. Below the surface the flowing silt-laden water forms a smooth deposit over the canal sides and bottom, and this is a factor that is partly responsible for the use of higher velocities of flow. The value of *n* in Kutter's formula would be under 0·020, if it were not for the aquatic growth; the actual values of *n* are probably between 0·021 and 0·029, and for design calculations 0·0225 is used.

Several structures have been built during the last 3 years in the main canals. Reinforced-concrete structures of ample capacity for the increased flow have been supplanting the earlier timber structures, though, in some cases, renewals have had to be made in timber. The characteristics of the larger canal structures such as checks are: a deep up-stream curtain wall of tongued or grooved sheet piling across the bed of the canal, extending up and well into the canal-banks; a wide floor, lengthwise with the canal, which is anchored to foundation-piles when the head on the check is large and when foundation conditions are not of the best; and a sheet-piling curtain wall, at the down-stream end of the structure to assure the safety of the structure from any bed-recession on the down-stream side. The line of the gates is placed at least 8 feet from the up-stream curtain wall, so that bed-scour will not occur in the canal above it. A footwalk extends across the structure just over high water and up-stream of the gates, to permit the ready removal of driftwood and floating debris that collects against the gates. The footwalks for operating the gate-lifting mechanism are usually on the down-stream side with the superstructure for road-bridges, when such are combined with the structure. Automatic radial gates are extensively used at checks and in wasteways, to maintain any constant water-level within a limited range by automatically controlling the discharges through the structures. One or two such gates are installed in a structure, and operate either singly or in unison for fluctuations of as much as 150 cubic feet per second.

Vertical drops, stepped drops with water-cushions, and inclined shoots constructed so that the kinetic energy of the falling water is practically destroyed by utilizing the principle of the hydraulic jump, are used to bring the water to a lower elevation wherever necessary. Salt Creek shoot has a capacity of 1,100 cubic feet per second and lowers the water 15 feet. At Rositas there is a single-cushioned vertical drop of 20 feet, and six stepped cushioned drops, amounting to 27 feet, giving a total drop of 47 feet for a capacity of 1,500 cubic feet per second.

The vertical-lift gates are of timber or steel. The lifting-mechanisms comprise: hand-wheel and screw-stem lifts, rack-and-pinion lifts operated by hand-powered gear and worm-wheel hoists, and lever bars for the small gates. Engine-driven hoisting-mechanism is used at Hanlon gate and at Hemlock sluiceway, where there is a head of 28 feet on the gate.

The soils are sedimentary silts of variable texture, and are of great depth. Having been desert ground formerly, they are devoid of humus and deficient in nitrogen, though the necessity for applying commercial fertilizers has not arisen, as potassium and phosphorus, the essential plant food elements, are present in sufficient quantities to render the soils permanently fertile. Many of them are compact and heavy, requiring much cultivation to maintain their proper physical condition. Plant growth and the ploughing-in of leguminous crops are causing the production of humus and so improving their fertility.

The great depth of subsoil was deposited in salt water, before the formation of the deltaic ridge which shut out the sea waters of the gulf; and it is claimed that the river-sediments sometimes have saline properties.

Continuous irrigation and seepage from canals for several years in some localities, where substrata of impervious silt-formed adobe soil exist, have produced a water-table within a few feet of the ground surface; and capillarity, coupled with the effect of a hot sun, has been the cause of a continual rise of alkali into the agricultural soils. In recent years, this condition has become very serious, and already over 120,000 acres are too much damaged for the economic production of crops. Field investigations, and a study of the problems to be solved for reclamation by drainage, have been made, and a programme of work, involving the dredging of a system of deep main drains, is shortly to be commenced.

WATER DUTY AND COSTS.

The quantity of water diverted into the canal-system each month since 1912 is shown in *Fig. 11* (p. 174), which illustrates the growth of the project. The decrease apparent for 1921 is due to the slump in the cotton market, resulting in a smaller area being cultivated for that year.

The rainfall for this arid region is less than 2 to 2½ inches yearly, and is mostly the result of isolated cloudbursts, being seldom general at any one time over the whole area. Agriculture is, therefore, entirely dependent on irrigation. The average gross duty for all crops for the 4 years 1916–1919 was 4·08 and 3·70 acre-feet per acre for the United States and the Mexican portions of the project respectively. The latter portion was confined chiefly to cotton-growing, while the former comprised diversified crop production. The largest of the water companies irrigated 114,850 acres for the year 1918, when a water duty of 3 acre-feet per acre was attained. The cost of the water per acre to this company in the same year was $5·52. The average cost of water delivered for the 5 years, 1916 to 1920, was $5·58 per acre for the whole United States portion of the project. For 1920 the cost per acre for this portion of the project was as high as $7·67, which is a maximum, for it will be lower in subsequent years. To the cost of water a farmer must add $2 to $3 per acre for his state and county taxes, thereby making a total liability of $8 to $10 per acre to be added to the cost of ploughing, seeding, and cultivation.

The Author is indebted to Mr. F. N. Cronholm, General Manager of the Imperial Irrigation District, for permission to use the records which have enabled the Paper to be written. Mr. Cronholm is the originator of the type of diversion-weir shown in Fig. 12, Plate 2.

The Paper is accompanied by seventeen sheets of tracings, from some of which Plate 2 and the Figures in the text have been prepared.

(*Paper No. 4425.*)

"The Influence of Silt on the Velocity of Water Flowing in Open Channels."

By Arthur Burton Buckley, Jun., O.B.E., Assoc. M. Inst. C.E.

The fact that silt has an influence on the coefficient of friction in rivers and canals has been known for some time, but its full significance appears to have escaped observation. Indian experience shows that variations in the coefficient of friction have been commonly observed to accompany deposits of silt. The most definite reference to the phenomenon appears, however, to be due to Mr. Duponchel, who, in referring to certain experiments carried out by him on the flow of water in a small artificial canal, says[1]:—

> "It may be stated as a fact, proved by experience, that running water, having a mean velocity of 1·44 metre per second, can carry 9 per cent. of its own weight of silt; and as, moreover, the admixture of silt, far from having diminished the velocity, appears rather to have increased it, it may be concluded with very great probability, that this proportion of 9 per cent. is well below the degree of saturation which accompanies a velocity of 1·44 metre."

Mr. A. Flamant, in quoting this passage, adds: "In fact, it appears from these experiments as if, other things being equal, the mean velocity of water charged with silt is greater than that of clear water." Mr. Flamant discusses various contrary opinions put forward by other experimenters, but does not commit himself to any agreement with them; in fact, he declares them to be, in his opinion, inconclusive. His discussion of the subject affords an interesting and instructive example of accurate thinking in connection with an obscure subject, and he concludes by emphasizing the importance of further investigation.

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[1] "*Hydraulique et géologie agricoles,*" pp. 638–639 (quoted in A. Flamant's *Mécanique appliquée hydraulique*, p. 306. Paris, 1891).

The phenomenon is so intimately connected with the theory of hydraulics that it is impossible to omit reference to the work of Messrs. Ganguillet and Kutter, published in 1869. To the names of these two distinguished Swiss engineers must be added those of Messrs. Rudolph Hering and John C. Trautwine, jun., the translators, in 1888, of Kutter's publication. These engineers must always hold the first place in the field of hydraulic research for having compiled a collection of statistics which has been of the greatest assistance to others. The English translation of Kutter's work contains allusions to the subject of this Paper. The Authors were endeavouring to arrive at a general formula for the flow of water with a coefficient of roughness depending on the nature of the bed, but in compiling their tables of 1,200 gaugings, with deduced values of the coefficient of roughness, they had already observed, without being able to account for it, the fact that this coefficient might vary very considerably in the same channel.

In 1918, a series of current-meter gaugings of the discharge of the Nile was begun in the Khannaq reach of the river, some 28 kilometres north of Assuan Nile gauge. Careful Nile gauge-observations were made for various reaches simultaneously with the velocity-observations, and silt-observations also were carried out, at various times, in order to throw light on a branch of hydraulics which appeared to have been previously somewhat overlooked.

In the course of the plotting and general scrutiny of the Nile discharges about to be discussed, certain abnormal fluctuations in velocities were remarked which appeared to require explanation, and these were discussed in a preliminary manner in a departmental paper dated the 17th August, 1920. Owing to the complexity of the problem, it was difficult to draw definite conclusions from the results as they then existed. The accuracy of the results depends on the simultaneous observation of several variable quantities, and it was felt at that time that owing to many complications it was difficult, without further corroboration, to place sufficient reliance on the data collected to warrant the enunciation of what was believed to be an original theory.

Meanwhile, considerable progress was being made in the development of appliances in connection with a mechanism known as the Buckley-Wilson discharge-recorder, designed with the object of indicating automatically the discharge in the Menufia canal, which irrigates nearly a million acres in the Delta. In this instrument two of the above-mentioned variables are mechanically combined, and the personal element is therefore partially eliminated. It was thought that the behaviour of the recorder, when working with clear and with silt-laden water, might help to throw more light on the problem. For instance, as the recorder was designed with a constant coefficient of roughness for the channel under clear-water conditions, it was thought that it might show an error in the discharge if the presence of silt in the water did, in fact, influence that coefficient. A detailed description of the recorder is given in Appendix A, (p. 198).

Fig. 1, Plate 3, presents a number of data collected in connection with the instrument. The curve denoting error per cent. indicates the discrepancy between the discharge, as shown by the dial of the instrument at a specific time of the day, and the discharge according to the official records; and it will be seen that from the 30th June, when the instrument started to work, to the 18th August, the error was of the order of 1 or 2 per cent., although the slope of the canal had varied from 4·8 to 8·2 centimetres per kilometre and the discharge had varied from 200 to 300 cubic metres per second.

Within 2 or 3 days from the 18th August, the error of the machine increased to 13 per cent. in defect, with a drop in the slope to an average value of 5 centimetres per kilometre. In Fig. 1 is given a curve showing the silt in suspension in the water flowing in the Menufia canal and in the Nile from which the canal takes its supply. A peak in the Menufia canal silt-curve appears on the 27th August, accompanied by a corresponding peak in the error-curve towards the line of zero error.

It is evident that the reduction in slope and discharge which began on the 18th August, and which moreover synchronized with the period of maximum silt in suspension, must have caused precipitations of silt in the canal. On the other hand, the increase in slope and discharge recorded from the 22nd August would have the effect of picking up the silt previously precipitated, this being shown by the peak in the silt-curve of the Menufia canal on the 27th August, with the corresponding peak in the error-curve towards the line of zero error. Hence the error in defect of the recorder coincides with the silt-precipitations which can be expected to occur with the conditions just described. It will be seen that by the 15th September the error-curve of the recorder had returned to the line of zero error, by which time the slope and discharge had again been increased. The error-curve from the 15th September to about the 20th October is seen to fluctuate between errors in excess and in defect, which neutralize each other; these are fully discussed in Appendix A, and do not influence the main argument.

Hence it appears that, on a certain date after the arrival of the silt-laden water, the recorder showed an error of defect, indicating that the coefficient of roughness in the canal had diminished. This error, however, did not persist. It existed during the period when the slope was lowered abnormally, causing silt-precipitation, and it disappeared when the slope was increased. It appears, therefore, that the silt has a lubricating effect which may be explained as follows.

A mass of water starting from rest—as, for instance, on issuing from a lake—if flowing down a channel of uniform slope and section, assumes a condition which, with sufficient accuracy for the present discussion, may be described as steady flow; and its behaviour in this respect differs from that of many other travelling masses whose motion is equally due to the influence of gravity and which undergo acceleration. The friction between the water and the periphery of the channel is the cause of retardation. Anyone who has observed the behaviour of the very light particles of sand found sometimes at the bottom of clear streams will have remarked the eddies which exist. The retardation of the flow alluded to is brought about by the work expended in friction of the mass of water against the channel in passing from one level to another, and this work takes the form of eddies. When, therefore, a stream carrying solid matter in suspension has its velocity arrested, a precipitation of the solids occurs. The result is the interposition of a much heavier fluid, or slurry, between the principal mass of water and the periphery of the channel. The energy of the water, while sufficient to create eddies in the clearer water, now becomes insufficient to produce the same effect on the heavier liquid, with the result that the eddies, which before were communicated far towards the heart of the channel, now become damped; the retarding effect due to the friction with the periphery is therefore partially eliminated, and the water acquires a higher velocity.

The foregoing conditions must have been set up between the 20th and 30th August in the Menufia canal. But with the increase in slope the water must have again picked up the silt which it had begun to deposit on the 18th August, and, indeed, the peak in the Menufia silt-curves on the 27th August is evidence to this effect. In picking up the silt, the water removed the " blanket " of slurry which had damped the eddies, the coefficient of friction of the channel returned to its clear-water value, and, by the 15th September, the error of the recorder had disappeared.

Evidence therefore points to the precipitation of silt as the cause of this phenomenon, though the presence of silt in suspension need not necessarily alter the coefficient of friction of a channel. And, further, it appears that the effect of the precipitation has itself a limit, which is reached when the silt is allowed to solidify, as, for instance, when a canal is allowed to dry, for the slurry then ceases to exist, and, when the water is again admitted, it finds a newly solidified surface against which to expend its energy. This effect has been traced through the error of the discharge-recorder, and phenomena of a similar nature have been noticed in the Nile after a sufficient number of discharge-observations had accumulated.

Experiments at Baqlawis Nile Discharge-Station.

The methods adopted for observing the discharges are described in Appendix B (p. 202), Fig. 2, Plate 3, represents the various hydraulic elements obtained at the Baqlawis discharge-station for 1919. One of the first indications that something peculiar might be expected was given by the sudden drop in the mean velocity of the river on the 12th, 13th, and 14th October: this is clearly shown by the velocity-curve. The programme of work included the plotting of all data both against the Nile gauge and against time. The behaviour of the sectional area of the trough of the river (Fig. 3, Plate 3) heightened the interest already aroused. This curve represents the section of the Nile at the discharge site, plotted against the Nile gauge at the site, and shows that, whereas above R.L. 86·20 the sectional area for the falling stage departed considerably from the curve for the rising stage, on the 14th October it assumed a pronounced change of direction towards the rising curve. It was on the 13th October that the sluices were first adjusted to start the filling of the Assuan reservoir. The agreement between the values of the cross section for the rising and falling periods below R.L. 86·20 is too complete to leave room for doubt that the conditions of the bed were identical before the 7th August and after the 15th October, the dates on which the water-level passed R.L. 86·20 when rising and falling.

The next step was to scrutinize carefully the observed velocities of the water on the 12th, 13th and 14th October. Fig. 4, Plate 3, shows the peripheral velocities at a height of 1 metre above the bottom, and the profile of the river-bed on the 12th and 14th October. It is evident that, in every case where scouring occurred, the peripheral velocity was diminished. A very interesting feature appears, namely, that at buoy No. 7, where, on the 14th, silting had occurred, as shown by the hatched area, instead of the scour which occurred generally all over the rest of the section, the peripheral velocity was increased. It appears, therefore, that a purely local variation in the coefficient of friction may occur independently of the general tendency of the bottom in this respect. It is not suggested that the solidified deposit was the active agent in reducing friction; the presence of this deposit would point to the existence of a liquid heavier than the rest of the water in the river, which was probably the real agent.

Experience has established that the velocity of water flowing in an open channel may be expressed by the following general formula :—

$$V = c.f(R).f(S),$$

which usually takes the following form : -

$$V = c\sqrt{RS}.$$

where V denotes the velocity in metres per second, R the hydraulic radius in metres, and S the sine of the angle of slope of the water-surface.

The value of the coefficient c, although supposed to be definite for any definite channel, has always been the subject of much discussion. It will be shown that the same value cannot always be applied to it, even for the same values of R and S. It is therefore now emphasized that the diminution in velocity before referred to was greater than could be due to a reduction in the value of the hydraulic radius. The elements are given in the following Table :—

Date.	Mean Velocity.	Slope.	Hydraulic Radius.	Level of River.
	Metre per Sec.	Cm. per Km.	Metres.	R.L.
October 12, 1919 .	1·215	6·88	8·23	86·69
„ 14, „ .	1·078	6·90	8·04	86·36

Decrease 2 per cent. in Hydraulic Radius.

The slopes are computed on the Assuan gauge and on the gauge at site. The influence of the small increase in slope on the 14th October makes no measurable difference, and, while a reduction of 11 per cent. occurs in the velocity, a reduction of only 2 per cent. takes place in the hydraulic radius.

The variations in the ratios of the peripheral velocity and the maximum velocity in the section on the 12th and 14th are significant of the altered peripheral friction. They are as follow :—

$$\text{October 12} \quad . \quad . \quad \frac{\text{Maximum velocity in section}}{\text{Peripheral velocity}} = \frac{1 \cdot 75}{1 \cdot 05} = 1 \cdot 67$$

$$\text{,, 14} \quad . \quad . \quad \frac{\text{Maximum velocity in section}}{\text{Peripheral velocity}} = \frac{1 \cdot 60}{0 \cdot 87} = 1 \cdot 83$$

In other words, the velocity nearest the bottom of the channel was relatively higher on the 12th, when the channel was silted, than it was on the 14th, when it had scoured; showing that an increase of velocity had occurred in that stratum of water which is immediately influenced by the friction of the bottom.

The evidence appeared to be of considerable interest, and, although discussed in the departmental paper already mentioned, the theory could only be advanced tentatively. There appeared to be no reason for doubting the accuracy of any of the data, which had been selected by conscientious and reliable observers; but the fact remained that the results might have been vitiated by any error which might have occurred in carrying out any of the observations. The fact that any variation in one variable was generally accompanied by an unexpected variation of unknown magnitude in the other two, greatly complicated the study. It has already been explained how confirmatory evidence was obtained from other experiments carried out with the help of the discharge-recorder, which, by mechanically combining two of the variables, eliminated several possible sources of error.

In 1920, with a view to collect data for the elaboration of an electrical power-scheme in connection with the Assuan reservoir, it was decided to regulate on the dam during flood so as to increase the natural head of 2 metres up to a maximum of 5 metres; and every effort possible within the limits imposed by financial restrictions was made to observe any effects on the river which might be produced by the altered conditions thus set up. As the variations in cross section of the river alluded to on p. 187 were not understood, and might have indicated that the site chosen for observing the discharge was possibly not the best, it was decided to transfer the discharge site to a point 1,500 metres down-stream, known as the Khannaq site.

KHANNAQ STATION.

Fig. 5, Plate 3, shows a portion of the hydraulic elements for 1920 plotted against the Nile gauge. To facilitate reference, points on the curves which are alluded to in the text are distinguished by their dates. At the crest of the flood peak all the hydraulic elements were continually varying. The sectional area, however, reached a state of fair equilibrium on the 16th September, having the same value as during the rising stage at the same level, while the slope was the same (reference line WX, Fig. 5). The reading of the Khannaq gauge on the 16th September was 87·54. During the rising stage this level was reached on the 16th August, when the silt in suspension amounted to a little less than 2,000 grams per cubic metre, whereas, with the same level on the 16th September, it only amounted to 1,000. Fig. 6, Plate 4, shows the curve of silt in suspension. The drop from the 16th August to the 16th September is clearly indicated. It shows, moreover, that the reduction in silt coincides with the reduction in sluice-area from 1,490 to 1,257 square metres.

Examining the curve of mean velocities on line WX, in Fig. 5, Plate 3, it is evident that the velocity was reduced, from the rising to the falling stage, from 1·48 to 1·32 metre per second, or by 11 per cent., the gauge, slope, and sectional area being the same—as shown by the points in the respective curves where line WX crosses them—while the silt was notably different. It can reasonably be concluded, therefore, that the reduction in velocity was brought about by increased friction.

Again, between the observations of the 17th and 19th August a notable drop in velocity occurred (Fig. 5, Plate 3). This drop coincided with the closing of the sluices, as shown by the drop in the sluice-opening curve in Fig. 6, Plate 4, where the ordinates are also dated. In fact, a repetition occurred of the phenomenon already described for the 12th and 14th October, 1919, namely, another instance of considerably decreased velocity with constant slope and an 8 per cent. increase in cross section of the river with a reduction of sluice-openings and increased water-level, but with a reduced discharge.

The peak in the sectional-area curve dated the 28th July (Fig. 5) affords another example. The diagram shows the area of opening of the sluices in the dam corresponding with each date. It shows that the sluice-area was increased steadily with a practically constant reservoir-level (Fig. 6) from 552 square metres on the 22nd July to 682 square metres on the 26th July ; this was followed by a reduction on the 27th. On the 28th the sluice-area was still below that recorded for the 26th. Finally, on the 30th July, the sluice-area was again increased and brought to 764 square metres. A portion of the Halfa gauge-curve is plotted in Fig. 6. It shows a check in the rise of the river, which coincides with the same check at the Assuan gauge and with the same sluice-reduction shown in Fig. 5.

The latter, although in itself relatively small, when combined with the check in the rising levels, can be expected to have produced silt-precipitation in the reservoir. The increase in the sectional area of the river dated the 28th July in Fig. 5 coincides exactly with this period. On the 28th there was a slight decrease in the discharge with a constant level, in spite of the increase in the river's sectional area; the slope was also slightly increased, but the velocity was reduced.

The readiness with which the section of the river responds to alterations of the sluice-area in the Assuan dam can be shown in a striking manner with the help of Fig. 3, Plate 3. On the 2nd October certain sluices in the dam were closed, and the reservoir-level was raised temporarily 0·30 metre for scouring silt-deposits from the locks. By the morning of the 4th all the sluices had again been opened. Fig. 3 shows that after the 3rd October the sectional-area curve took a marked turn, showing an increased sectional area in spite of a drop in the river-level of 0·22 metre. The reduction of the velocity in the reservoir by holding up the level at a time when the silt-content in the Nile was 1,400 grams per cubic metre cannot have failed to induce silt-deposits in the reservoir. The hump in the sectional-area curve, Fig. 3, which culminated on the 7th October, was probably caused by silt scoured out from the reservoir owing to the re-opening of the sluices, which was deposited at Baqlawis and, no doubt, elsewhere. The distance of 31 kilometres from the dam to Baqlawis would account for the fact that no increase in sectional area was recorded at the latter station until after the 3rd October. Unfortunately, observations are lacking for the 4th October, for possibly the section on that day might have thrown further light on the subject. Finally, as already explained on p. 187, when on the 13th October the sluices were first adjusted for filling the reservoir, the effect of this operation was felt at Baqlawis on the morning of the 15th, as shown by Fig. 3.

The evidence afforded by these results appears to be very significant. During flood, the river, in its natural state, undisturbed by the action of the Assuan dam, flows with a stratum of heavily-silted water along its bed, producing the blanketing effect already described. The checking of the velocity of the river in the reservoir by regulating on the sluices must cause silt-precipitation in the reservoir. Therefore, the water, issuing from the reservoir with less silt than it would otherwise be carrying, picks up silt downstream of the dam from the heavy layer near the bottom, thereby destroying the friction-reducing agent. In fact, at Baqlawis in 1919, and at Khannaq in 1920, there was set up the reverse effect to that which produced the error of defect of the Menufia discharge-recorder; in that case the velocity in the canal was increased by silt-precipitations, with the result that the recorder, being designed with a coefficient of friction for clear water, gave too low a reading.

The hydraulic elements at Khannaq for 1921 provide further examples. In that year the slope during the falling period became greater than during the rising period, commencing from R.L. 86·95 on the 15th October. It gradually returned to the rising-period curve until at about R.L. 84·55 on the 21st November it again began to leave it. On the 17th October, when the slope-curve for the falling stage first began to leave the rising curve, the Assuan sluices were first manipulated, and a constant level was maintained in the reservoir. This would precipitate silt in the reservoir, and the increased friction in the river would call for the increased slope. Subsequently, when on the 21st November heavy regulation was commenced for the filling of the reservoir, the friction of the bed must have again been increased, keeping the slope of the river higher than it was during the rising stage, although the sectional area for a given level was greater, while the velocity was slightly less (Fig. 7, Plate 4). A particular feature of the curve of velocity plotted against gauge as ordinate is that, from the lowest level (R.L. 82·00) to a point dated the 4th August, R.L. 84·35, the curve has, for all practical purposes, a constant direction, which takes an abrupt turn on that date, indicating a sudden increase in acceleration. Silt-determinations at Halfa (Fig. 8, Plate 4) showed that the arrival of the silt dated from the 28th July; allowing 3 or 4 days from Halfa to Assuan, the two dates are found to correspond. The same effect can be traced with less distinctness for other years, and for convenience of reference the dates are tabulated:—

Year.	Estimated Date of Commencement of Rise of Silt-Curve at Halfa.	Date of Critical Point in Velocity-Curve.
1919	July 23	July 30
1920	July 14	July 13
1921	July 28	August 4

Beleida, 1921.

In 1920, another discharge-station was established at Beleida, 38 kilometres up-stream of Roda gauge and 832 kilometres from Khannaq. It was equipped as at Khannaq. A complete series of silt-determinations was obtained for this station, twenty-four samples being taken for each determination at six buoys anchored in the Nile. Appendix C (p. 204) gives descriptive details of the methods employed in dealing with the silt-samples. The slopes recorded are those computed on the site gauge and on the gauge at Elessi pumping-station situated 14·5 kilometres up-stream of Beleida. The slopes at this discharge-station are influenced by regulation on the Delta barrage, which lies 65·5 kilometres down-stream. Actually the variations in slope which can be obtained are limited by the level which must be maintained to feed the canals, which derive their supplies at the barrage; but, during the period from about the 25th December to the 5th February, when canals are closed for clearance of silt, the up-stream level at the barrage is lowered: this is the explanation of the increased slope shown at the end of January in Fig 10, Plate 4. The slopes from the 8th to the 25th February are not accurately representative of the conditions of slope of the river, owing to rapid variations brought about by a wave liberated from the Assuan reservoir to benefit navigation at that particular time.

Fig. 10, Plate 4, shows the first maximum slope peak on the 13th August, with a slight tendency to drop up to the 22nd August, and Fig. 9, Plate 4, shows a portion of the sectional-area curve for the corresponding period. The hydraulic elements are as follow:—

Date.	Velocity.	Hydraulic Radius.	Slope.
	Metre per Sec.	Metres.	Cm. per Km.
August 13, 1921 . .	0·884	5·36	8·0
„ 22, „ . .	1·242	7·28	7·6

Increase 35·8 per cent. in hydraulic radius.

In order to compare the velocities on the two dates, they should be referred to the same slope. Thus on the 13th, other conditions being the same, the velocity with a slope of 7·6 centimetres per kilometre would have been 0·862 metre per second. With this reduction the velocity of 1·24 observed on the 22nd represents an increase of 43·8 per cent. The increased velocity due to increase in the hydraulic radius would not have exceeded 35·8 per cent., leaving a net increase of 8 per cent. accountable for by the reduced friction in the channel. The silt began to increase on the 1st August, and by the 13th August the river was carrying 700 grams per cubic metre. The increase in velocity of 8 per cent., just alluded to, coincides with the arrival of the silt-laden water.

EFFECT OF WATER-LEVELS.

A variation in the coefficient of roughness for a given discharge would influence the water-levels in the channel; an example of this occurs in the case of the Ibrahimia in Upper Egypt, commanding 408,000 acres at the reach considered. The following hydraulic elements have been extracted from the Irrigation Records:—

Date.	Gauge.	Slope.	Velocity.	Hydraulic Radius.	Area.	Discharge.
		Cm. per Km.	Metre per Sec.	Metres.	Sq. Metres.	Cu. Metres per Sec.
August 16, 1917 . .	42·3	6·63	0·907	3·29	162	147
„ 11, 1918 . .	42·55	6·69	0·764	3·43	174	133

Decrease in discharge 10 per cent.

It will be seen that, whereas in 1918 the slope was slightly greater and the level 0·25 metre higher, the discharge was 10 per cent. less than in 1917. The extra sectional area in 1918 has been verified and found to correspond with the increase in water-level—that is, the profile of the canal was the same on both occasions. Silt-statistics being lacking, this example simply affords evidence of the fact that in 1918, although the canal was running at a level 0·25 metre above that of the previous year, it was carrying 10 per cent. less discharge instead of about 11 per cent. more, as might have been expected from the increased area and hydraulic radius.

The Menufia canal, already referred to in connection with the recorder, irrigates between 900,000 and 1,000,000 acres in the Delta. The 18th to the 27th August was the period of silt-precipitation, the level at the head gauge was 15·40, the slope was 5 centimetres per kilometre, and the measured discharge was 258 cubic metres per second. On the 25th November, when silt had practically disappeared from the water, with the same level and slope the measured discharge was only 214 cubic metres per second, or 17 per cent. less than in August. In order to discharge 258 cubic metres per second in November, it would have been necessary to raise the canal by 0·53 metre.

CONCLUSION.

At the beginning of the Paper, reference was made to Kutter's coefficient of roughness *n* and to the variations in its values which the compilers of the later edition of Kutter's publication had discovered to exist in the same channel. The value of this coefficient was computed for Beleida and is plotted in Fig. 10, Plate 4. At a first inspection it appears that the coefficient decreases with the increase in the silt and remains fairly constant during the months of August and September, subsequently again increasing with the diminishing flood. Examination showed that, although it might be possible to find a simple relation between the coefficient and the silt in suspension which would apply to the months of August and September, when the slope of the river was subject to small variations, the relation broke down when applied to the rising and falling periods. One of the Author's assistants, Mr. S. Lelyavsky, was therefore entrusted with the examination of the problem, and he arrived at the following formula, known as the Beleida formula, by the process described in Appendix D (p. 207):—

$$V = \{147 + 3{\cdot}92\ (Z - 10)^{0{\cdot}383}\}\ R^{0{\cdot}85}\ S^{0{\cdot}72}$$

where V denotes the mean velocity in the river in metres per second,
Z denotes the average amount of silt in suspension, expressed in grams per cubic metre of water,
R denotes the hydraulic radius in metres,
S ,, ,, sine of the angle of slope of the water-surface,
and 10 is the minimum silt-content during the low stage, in grams per cubic metre of water.

From January to June, when the silt in suspension was at a minimum of about 10 grams per cubic metre and when special samples were not collected at Beleida, the Public Health Department Laboratory determinations have been used. The velocities computed by this formula are given in Fig. 11, Plate 3. It will be seen that the agreement between them and the observed velocities is very close. For the sake of comparison the velocities have also been computed by means of Kutter's, Bazin's, and Manning's formulas, and these are plotted on the same diagram.

$$V = \frac{23 + \frac{1}{n} + \frac{0{\cdot}00155}{S}}{1 + \left(23 + \frac{0{\cdot}00155}{S}\right)\frac{n}{\sqrt{R}}}\sqrt{RS} \qquad n = 0{\cdot}057 \text{ (Kutter).}$$

$$V = \frac{87}{1 + \frac{\gamma}{\sqrt{R}}}\sqrt{RS} \qquad \gamma = 3{\cdot}98 \text{ (Bazin).}$$

$$V = \frac{1}{n}\sqrt[3]{R^2}\sqrt{S} \qquad n = 0{\cdot}045 \text{ (Manning).}$$

All three formulas contemplate the use of a fixed coefficient, which, in this particular instance, has been determined for the clear-water period. It is evident that for all practical purposes they are of very little value, for the coefficient is not constant, and Fig. 11 clearly shows that, by making no provision for this variation, all three formulas give velocities which are as much as 50 per cent. in error during the flood period. It would appear, therefore, that no satisfactory formula has been discovered in the past, owing to the neglect of an apparently important factor.

For testing the general applicability to other Nile discharge-stations of the general principles on which the Beleida formula is based, silt data taken simultaneously with the discharge-observations should be available. Silt-determinations are available for Khannaq in 1920. The samples were, however, taken at only two buoys, anchored at each end of the middle third section of the river, instead of at six buoys as at Beleida. Only a rough test is therefore possible, which may be applied by computing the velocities at Khannaq, using the Beleida formula as it stands. The results when compared with the observed velocities show a close agreement in principle (Fig. 12, Plate 4.)

With the limited information available, more definite conclusions cannot as yet be arrived at; nevertheless, owing to the concordance of the results obtained by applying the Beleida formula to the Khannaq discharge-station, it would appear that further investigation would lead to the discovery of constants in the formula, rendering it applicable to many reaches of the Nile and giving precise results.

The absolute differences which occur between the observed velocities and those computed by the Beleida formula are of an order which appears to justify their being attributed to lack of precision of observation. The magnitude of the scale of the experiments must not be forgotten, as well as their cost. More precise means of observation imply the application of laboratory methods to the measurement of the discharge of one of the largest rivers in the world. They imply continuous instead of single daily records of all the quantities, with the erection of complicated observing- and recording-instruments. The cost of equipping such a station would amount to several thousand pounds, which appears to be beyond the present available resources.

It might have been more satisfactory to delay this communication until more experience had been accumulated; but, on the other hand, owing to the nature of the phenomena, which occur in cycles of 12 months, it would appear that it must necessarily be several years before any such results can be hoped for in Egypt, apart from the difficulties which have arisen out of the critical financial situation, which obtains there just as much as elsewhere. Moreover, it appears almost essential to obtain statistics for several different countries, where the silt carried in suspension is widely different, before any general conclusions can be arrived at. It appears to be the consistency of the slurry at the bottom of the channel which influences the friction, and not the amount of the silt actually in suspension, and it may not follow that the relation between the two, as expressed by the Beleida formula for Egyptian conditions, will be the same in other countries. The revision of the Kutter-Trautwine statistics, which embrace Belgium, France, Germany, Holland, Italy, India, Russia, Switzerland, and the United States, would therefore appear to be indicated.

In the computations in connection with the Paper, the Author has been assisted by Messrs. J. Schoyen, S. Lelyavsky, J. Massara, Mohammed Amin, Yusef M. Semeika, and Ahmed T. Topozada. The operations in the field were under the direction of Mr. A. W. Wylie, assisted by the above and a staff of observers for discharge-operations and Aly Bey F. Saad-el-Din and Faik Neguib Ibrahim for silt-collection.

The Public Health Laboratories' silt-determinations were available through the courtesy of Dr. C. Todd, O.B.E. The determination of the silt-content of the samples specially collected in connection with the subject of the Paper was under the direction of the late Mr. F. Hughes, Chemist to the Ministry of Agriculture, whose untimely death is very much regretted.

Thanks are due to Major G. F. Schreiber, R.E., Inspector-General of Telegraphs, Captain W. J. Hilyer, Assoc. M. Inst. C.E., and Mr. A. W. Ayscough of the Department of Telegraphs and Telephones, for expert advice in connection with the electrical transmission-lines for the recorder; also to Mr. L. Bertoni for the general supervision of the instrument.

Mr. J. S. Wilson, Assoc. M. Inst. C.E., has throughout been intimately connected with the design and construction of the recorder. The conception of the electrical transmitters, without which the recorder was never a working instrument, is due to him.

The Author's thanks are due also to many other officers of the Public Works Ministry who have assisted in the work.

The Paper is accompanied by one map, twenty-two tracings, and one photograph, from some of which Plates 3 and 4 and the Figures in the Appendixes have been prepared. It is also accompanied by Tables of all the hydraulic elements derived from 480 gaugings at Khannaq and Baqlawis and 192 at Beleida, and by Tables of 624 silt-determinations at the Beleida Nile gauging-station.

[APPENDIXES.

APPENDIXES.

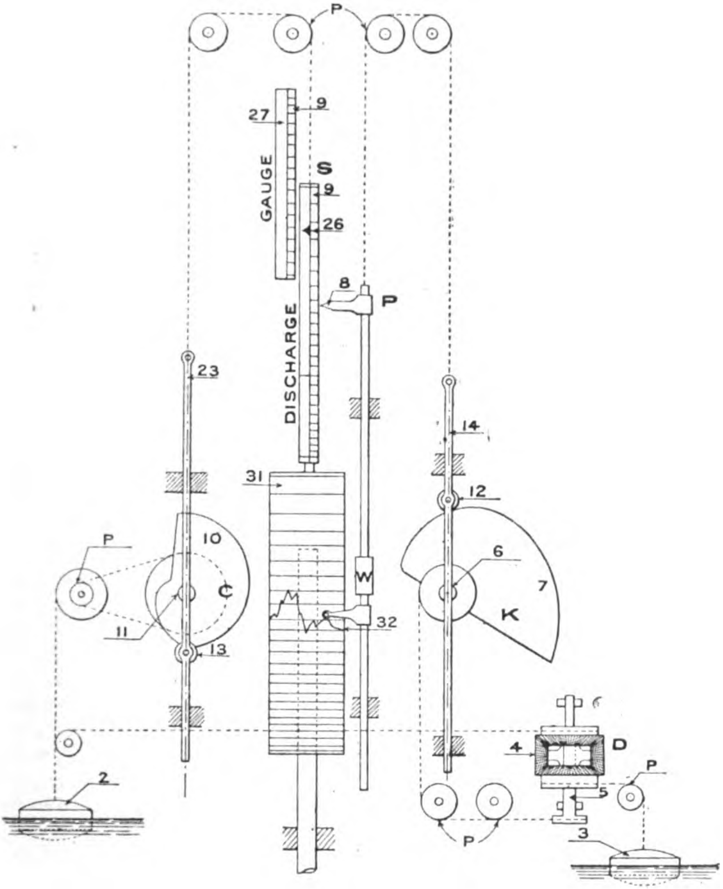
APPENDIX A.

BUCKLEY-WILSON DISCHARGE-RECORDER.

This instrument combines within its mechanism the level of the canal and its slopes, and permits the discharge being read off the dial or being recorded on the revolving cylinder as a weekly diagram. In *Fig. 13*, S is a scale graduated to represent the discharge of the canal at its mean slope. P is a pointer against which the discharge is read. The recorder is actuated by the motion of two floats erected in wells which, in the present case, are 5 kilometres apart on the left bank of the canal, the recorder itself being 2 kilometres from the upstream float. C is a cam actuated by an electrical transmission from the upstream float. D is a differential gear, one side of which is operated from the upstream float, while the other side is operated from the downstream float, with the result that the neutral member 4 of the gear can be made to indicate the slope of the water-surface of the canal. K is a second cam embodied in the recorder and mechanically connected to 4, the neutral member of the differential gear. The mechanism of the recorder is such that the pointer P is made to rise or fall in accordance with the amount of slope of the water-surface and thereby to introduce the necessary correction whenever the slope departs from the mean for which the scale S is graduated. The absolute variation in discharge for any given slope-variation is different for every level of the water-surface in the canal, but the ratio of one discharge to the other is the same for every level in the canal. In other words the difference between the logarithms of the discharges at any two slopes will be the same for any level in the canal; and, by adopting logarithmic divisions for the scales, it follows that the travel of the pointer P will be the same for any given slope-variation, whatever may be the reduced level of the water in the canal. In operation, therefore, scale S and the pointer P assume a definite position relative to the water-level, and the pointer P, by further rising or falling, adds or subtracts the necessary correction to suit the varying slope, the discharge being finally read on the scale S opposite pointer P. In the instrument scale S is engraved in a circle concentric with the face.

The recorder was designed in 1914 on data collected in 1913, but its development was retarded by the fact that no reliable means existed for connecting it with the moving floats. The consequence was that, by missing signals, its mechanism got out of step with the floats. The indications were frequently quite false and completely unreliable. Experiments had to be suspended during the war. On peace being declared the studies were resumed. An entirely new type of electrical transmitter was invented, which was independent of the breaking of the electrical circuit. After any period of interruption it would transmit as many signals as would have been transmitted had no interruption occurred. The whole system is such that the recorder was either in complete accord with the floats or else ceased to give indications, giving at the same time a warning to that effect. The transmission-line consisted of bare wires carried on poles. At the recorder end it passed through thickly-wooded grounds and was moreover menaced by contacts from several ill-arranged old telephone lines, with the result that some interruptions occurred which, for various reasons, could not be repaired as quickly as might have been desired. These particulars are given to explain the periods of interruption which will be found in the series of records.

Fig. 13.



DISCHARGE-RECORDER.

Fig. 1, Plate 3, contains the curve of water-slopes as indicated by the recorder, and a curve of percentage error between the recorder discharges and those measured either by current-meter or from curves of calibration for the sluices which, when the head on the regulator is not too small, can be relied upon to give results agreeing within 2 or 3 per cent. of current-meter determinations. Particulars as to the accuracy of current-meter discharges will be found in Appendix B. During the periods of low heads, current-meter discharges were taken at very frequent intervals.

The recorder was started on the 30th June. The time during which tests are available can be divided into six periods. During the first period the discharge varied from 200 to 300 cubic metres per second, with slope-variations from 4·8 to 8·3 centimetres per kilometre, while the magnitude of the error was negligible. The second period is dealt with on p. 185. During the third period the error fluctuated, being sometimes in excess and at others in defect. The curve of errors gives the value indicated by the recorder at 6 a.m., compared with the official discharges at that hour. With the conditions of heavily-fluctuating levels indicated by the gauge-curves the recorder and official gaugings are not strictly comparable, for, whereas the latter indicate the amount of water entering the reach through the head sluices, the recorder indicates the amount of water being taken by the canals. The reach is, in effect, a reservoir with a sloping surface, and a true comparison can only be made by integrating both curves for any given period and, in the case of the recorder, by adding or subtracting the amount of water represented by any difference in level found to exist between the beginning and the end of the period. The errors shown by the curve are therefore not true errors.

During the fourth period the electrical transmission-line was broken down. During the fifth period the error was negligible. During the sixth period, when the slope became very flat—of the order of 4 centimetres per kilometre—the recorder indicated high, and it is clear that, for these flat slopes, the radii of cam K required modification so as to reduce the discharge by a further 10 per cent. at 4 centimetres slope, decreasing to 3 per cent. at 5 centimetres slope.

As regards the second period, assuming the radii of the cam to be modified so as to make the instrument read an extra 3 per cent. lower with a slope of 5 centimetres per kilometre, it is evident that the total error indicated by the recorder would have amounted to 16 per cent.

It has been shown that, with the same level and slope on the 25th November, when there was very little silt in the water, the measured discharge in the canal was 17 per cent. lower than during the period mentioned in the previous paragraph, showing an exact agreement between the error which would have been indicated by the modified cam as alluded to above.

For the elaboration of the cams of the recorder in the first instance, use was made of Kutter's formula with 0·0275 as the value of the coefficient of roughness n. Subsequently, however, it was found that the formula could be expressed much more conveniently as follows, without any loss of accuracy :—

$$\text{Menufia Canal Formula } V = 14{\cdot}43\, R^{0{\cdot}71}\, S^{0{\cdot}402}.$$

Comparing this formula with the Beleida formula for the Nile, it will be seen that it makes no provision for an altered coefficient due to the presence of silt in the water. But it must be borne in mind that the conditions in the two channels are different. The slopes in the canal are to a certain extent under command, and hence every endeavour should be made to avoid setting up the conditions which tend to deposit silt and which call for the introduction of the silt factor in the formula. In fact, in the period after the 18th August, the error already alluded to in the recorder's indications no doubt arose from the fact that, as constructed, it does not include the silt coefficient in the formula. But, on the other hand, it appears that, by suitably manipulating the slopes, the conditions which call for the introduction of that correction need not arise in the canal, and care should be taken that they do not arise, if silting of the canal is to be avoided. In the Nile, on the other hand, the slopes are not under control: hence the necessity for the silt factor in the Beleida formula.

It will be seen that the recorder affords a direct means of determining when silt is being deposited in a canal. The recorder cams were designed on discharge-determinations based on maximum surface velocities observed by surface floats, using the following coefficients for reducing the maximum surface velocity to the mean velocity.

Hydraulic radius . .	2·0	2·5	3·0	3·5	4·0	4·5	5·0 metres
Coefficient	0·77	0·77	0·78	0·78	0·78	0·78	0·79

Given, therefore, a canal provided with a recorder during the months when the water is charged with silt, the development of a defective error in the machine, which can be detected by simple surface-float observations, affords a clear warning of silt-precipitation in the channel. Should the engineer in charge disregard this warning and fail to increase the slope of the canal, the silt would eventually solidify with the result that the error of the recorder would pass from a defect to an excess owing to the combined effect of increased friction and decreased water-section brought about by the solidification of the slurry. The engineer would then have a clear indication of the necessity for excavating the deposit from the canal. In a very short time the behaviour of every canal would be clearly known, and the conditions conducive to its silting would be precisely determined. As non-silting canals do occur in Egypt, accurate knowledge of the conditions which cause the silting of others should eventually lead to the diminution of this defect.

In the case of the Menufia canal it is now known that, with a silt charge of 1,750 grams per cubic metre of water and a gauge of 15·40 at the head, silt-precipitation occurs with a velocity of less than 1·100 metre per second. The determination of the silting-point of a canal can therefore be reduced to a matter of everyday routine within the capacity of a junior canal officer. Verification can be obtained at any time by his superiors through the inspection of routine registers.

The period from the 30th June to the 18th August (Fig. 1, Plate 3) affords an example of the comparative accuracy of the determination of the discharge of a large canal by two different methods, namely, by current-meter gaugings and by suface-velocity observations as embodied in the recorder. It is also of interest to note the stability of the canal's section, which has undergone no measurable change since 1913. The data collected show the degree of accuracy attainable in the measurement of discharges by slope-observations, which do not appear to have been given their proper importance in the past, possibly owing to the existence of certain misconceptions.

APPENDIX B.

ROUTINE FOR OBSERVING DISCHARGES OF THE NILE.

The discharges were taken by means of Price current-meters from motor launches fitted with the necessary appliances. Observers brought back the current-meters to the rating-tank after 75 hours' work. Current-meter observations were taken at intervals of about 50 metres across the river. Buoys were anchored in the stream in a line up-stream of the discharge-section, serving as convenient reference-points. The launches carried heavy anchors which were used at those points of the section where no buoys existed. The procedure was either to moor the launch to the buoy or to take the launch a sufficient distance upstream of the section and cast anchor. This having been done, the launch was allowed to drift down on to the discharge-section and was then aligned on to it by means of permanent beacons on the banks. The exact distance of the launch from the base was determined by angle observations made by box sextant on permanent beacons on each shore. The computations for determining the positions of the launch were made with the help of tables computed for each discharge-station, having the observed angle as one argument and the distance from the datum as the other.

The current-meter was suspended on steel piano-wire weighted with a conical lead weighing from 50 to 60 kilograms. The soundings were taken with the same lead after the lowest velocity-observation had been taken with the current-meter. A special stay wire, paid out from a fairlead at the bow and fixed to the point of suspension immediately above the current-meter, kept it vertically below the boat. During the flood season the taking of a discharge occupied about 6 hours, and during the low-water season about 2½ hours. For purposes of discharge-computation, the soundings were plotted on millimetre paper to a vertical scale of 1 : 100 and a horizontal scale of 1 : 1,000. On another part of the same sheet were plotted the observed velocities in each vertical of observation. The scale of depths was one centimetre to the metre, and the horizontal scale of velocities was 5 centimetres per metre.

The gauge-reading to which any discharge was finally referred was the mean of the readings taken immediately before and after the period occupied by the measurements of the discharge. All discharges are referred to a definite gauge, and at the Khannaq site simultaneous readings were taken on both sides of the river at the discharge-site as well as at other gauges 1,500 metres up- and down-stream. The observations of velocity were carried out in the usual way with the help of stop-watches.

The gauges consisted of tripods of 3-inch steel piping carrying one vertical limb, the whole being concreted into a solid base. To this limb was permanently fixed an angle bar, in one face of which were drilled a series of holes whose top edges were exactly 10 centimetres apart. At the top of the vertical limb was a grooved pulley over which ran a flexible wire rope passing inside the limb and terminating in a weight. The other end of the rope was attached to a micrometer device for reading the water-level.

The micrometer consisted of a gun-metal box 8 centimetres wide by 15 centimetres from back to front and 25 centimetres deep, sliding on the angle bar. The box was kept in place laterally by lugs cast at its four corners at the back. On the centre line half-way up the back of the box was a pin which engaged with the holes in the angle bar and fixed the position of the micrometer-box. The weight at the other end of the rope kept it in place.

In operation the micrometer-box is slid along the angle bar until it is partly immersed in the water. It carries a bridge-piece at the top, through which passes a screw about 40 centimetres long carrying a pointer which indicates on a circular scale. The pitch of the screw is 1 revolution per centimetre advance. An additional index travelling along a vertical scale enables the centimetre heights to be read. The bottom end of the screw is turned off square with a diameter of 20 millimetres, and it was found that the moment of its coming into contact with the water could be determined accurately by quite untrained observers. The square end was found more practical than a pointed end.

The micrometers were read by the ordinary watchmen, who were, however, selected men. Their instructions were to set the micrometers as already explained, to advance the screw slowly until contact was made with the water, and then to record the readings. The operation was performed from a boat, and the observer could easily see the sky reflected on the surface of the water in the micrometer-box. The top of the micrometer-screw was conveniently placed so that the reading of the gauge was reduced to a simple mechanical operation. For actually reading the scales, the micrometer could be lifted out of the water. In order to prevent oscillations of the water-surface the water was lead into the box through a flexible armoured tube about 3 metres long passing through the bottom. The meters and decimeters were read on an ordinary gauge fixed to the tripod.

The readings were therefore recorded in millimetres. This was in all probability a counsel of perfection, but the method ensured that the readings as recorded would yield accurate results with practically untrained observers. Moreover, the practical difficulty of reading the water-level directly inside the box had to be overcome, and the screw afforded the simplest solution. Besides aiming at accuracy, the whole arrangement was necessary to provide a waveless surface of water against the gauge. The usual method of constructing masonry wells at each gauge would have involved an expenditure far beyond available financial resources.

In order to check the accuracy of the discharges, a special experiment was carried out on the 19th January, 1919. Two parties worked simultaneously to observe the same discharge at sites situated 130 metres from each other. One determination yielded a discharge of 1,209 cubic metres per second and the other 1,233 cubic metres, the difference being therefore 2 per cent. Other comparisons made from time to time in canals also show that simultaneous current-meter determinations of a discharge, when carefully made, do not differ from each other by more than 2 per cent.

Rating Current-Meters in Silt-Laden Water.—When the irregularities of flow, which in the Paper have been attributed to variations in friction due to the presence of silt, were first noticed, a doubt arose as to whether the irregularities might not be due to the current-meters themselves, and the following tests were made, therefore, to detect, if possible, whether the rating-curves of current-meters rated in silt-laden water showed any departure from those obtained in clear water. The tests extended over 3 days. The first two were carried out at the Giza waterworks, where the open discharge from the pumps to the filters afforded a suitable channel of trapezoidal section 1·40 metre deep. At the top end of the channel was a considerable deposit of silt. To carry out the test a squad of men was detailed to stir up the silt as the pump was running, and, when the water was heavily charged, the pump was stopped, the outlet of the channel was closed, and the current-meter was rated in the silt-laden water. At the same time silt-samples were taken. and the silt-determination was carried out as usual in the laboratory, A number of observations were made with silt in suspension ranging from zero to 1,100 grams per cubic metre without any effect being detected.

As this test yielded negative results, it was determined to test the current-meter in water laden with an unmistakable excess of silt. For this purpose an iron trough allowing a clear run of 20 metres for the current-meter was used containing 2·9 cubic metres of water. Assuan pottery clay was obtained and ground by special machinery to an extreme degree of fineness. Sixty kilograms of this were added to the contents of the tank, the whole was stirred vigorously, and the current meter was rated. The silt in suspension was therefore of the order of 20 kilograms per cubic metre, but it produced no influence on the current-meter. From these experiments it was concluded that current-meters are not influenced by silt in the water.

APPENDIX C.

SILT RESEARCH.

The annual expenditure in the maintenance of Egyptian irrigation canals amounts to about £350,000. In order to find a means of reducing the expenditure, a campaign of silt research was organized in 1918 with the object of collecting information regarding the general conditions which lead to silting of canals. To understand them should mean their eventual control.

As it was not known to what extent the silt conditions in canals might differ from each other, and how the distribution of silt might alter in the river itself, a more detailed programme of silt-collection was arranged. Some sixty-three stations were therefore established all over the country, principally in the Delta, and the systematic collection of silt-samples was started.

The method of collecting the water is as follows:—The samples are collected in glass bottles containing 1,000 to 2,000 grams of water. Each observer is provided with a cylindrical wire cage, heavily weighted at the bottom, and attached to a sounding-chain by means of which the cage can be lowered into the water to the desired depth. Each cage is provided with an indiarubber cork, through which a bolt passes vertically, carrying a ring to which a line is attached. To take a sample the bottle is placed in the cage, the rubber cork is securely fitted into the mouth of the bottle, the bottle is lashed to the cage to prevent any danger of its floating out, and the cage is lowered into the water. When the cage has reached the desired depth the cord is given a sharp pull, and the cork comes away from the bottle, which fills with water at the particular level. When full the bottle is drawn up, securely corked, and placed in a padded box for despatch to the laboratory.

The method of collection has been found to give excellent results. Where relatively small quantities of fluid have to be relied upon, it is essential that the water should be subjected to as little handling as possible. The silt-carrying capacity of the water depends on its velocity, and it is therefore essential that the instant this factor is altered the water should be in the collecting bottle, and that it should not leave it until it reaches the laboratory. For this reason any method involving the collection of the water by pumping was viewed with distrust for fear of some of the silt being deposited in the tubing or other passages indispensable in such a method of collection. The observers have instructions to record the velocity of the water at the points where the samples are taken and to see that the corks are clean before being placed in the necks of the bottles.

The determination of the amount of silt in the water was described as follows by the late Mr. F. Hughes, Chemist to the Ministry of Agriculture, under whose direction the analytical work was done.

As a rule, by the time the samples reach the laboratory most of the suspended matter has settled out, and the water is only slightly turbid. Some 100 cubic centimetres of the surface water is removed by means of a pipette, and about 50 cubic centimetres of a 20 per cent. solution of ammonium chloride is added to each bottle. The volume of each bottle is marked on the bottle itself, which also bears a distinguishing number, so that no labels are necessary.

The addition of the ammonium chloride has the effect of causing the clay particles to adhere together into larger masses which quickly settle out, leaving the water quite bright. When settling has taken place in this manner, the clear liqud is removed by means of a siphon, care being taken that the sediment is not disturbed. By means of a fine but powerful jet of water the sediment is washed on to a 100 sieve fitted into a 200 sieve. The water which passes through the sieves is received in a large porcelain basin. A 100 sieve is one having 100 meshes to the linear inch. The sand collected on the 100 sieve is that called "coarse sand" while that passing the 100 sieve, but caught on the 200, is the "fine sand." The suspended matter passing the 200 sieve is the "silt and clay."

After washing, the sands are transferred to small porcelain crucibles, ignited, and weighed. The water containing the silt and clay is transferred to a beaker, and more ammonium chloride solution is added. After standing, the sediment is brought on to a filter, dried, ignited, and weighed. The weights are then calculated to parts per million, or grams per cubic metre. This mode of classification, while quite arbitrary, is justified by the following results obtained in some samples taken in the Nile, midstream, 300 metres upstream of the Rosetta barrage.

At certain depths samples were taken in duplicate, and the results, though not in perfect accord, when averaged and plotted, show a moderate regularity. The river at the point at which these samples were taken had a depth of 9·5 metres.

Parts per Million. Nov. 26, 1917.

Depth.	Coarse Sand.	Fine Sand.	Fine Silt and Clay.	Total.
Metres.				
1	3	32	324	277
2	3	60	352	415
2	8	78	354	440
3	6	113	368	487
4	6	174	351	531
4	15	127	363	495
5	15	160	355	530
6	20	163	377	560
6	20	169	384	573
7	24	237	393	654
8	42	266	394	702
8	47	200	386	633

It will be seen that, while the amount of coarse sand increases from the surface to the lowest sample nearly sixteen times, the fine sand increases less than seven times in amount, and the fine material—silt and clay—less than three times. Similar results have been obtained elsewhere, both in the river and in canals.

While it is fully recognized that the use of ammonium chloride may cause some slight alteration in the weights of the ignited mineral residues,[1] still, with a large number of samples, there seemed no other course possible; and as the results were required rather for comparison than for the determination of absolute quantities, there seemed to be no very serious objection to the employment of this precipitant. It should be noted that in all cases the figures refer to mineral matter. In no case has the suspended organic matter been determined.

Extensive records are now available for many canals in Egypt. It is beyond the limits of the Paper to describe them all in detail, but a few general conclusions are recorded. In a general way it may be said that the solids in suspension can be divided into two main categories, namely, sand which is carried along in the lower levels of the channels, and which is easily deposited, and fine silt which can, with a moderate velocity, be carried right away to the fields. The sand which travels along the Nile can be excluded from canals by building the sills of regulators at such a level that the water drawn into the canal is free of sand.

In 1921 efforts were concentrated at the Boleida Nile discharge-station to obtain records of the amount of silt in the river. They were taken once a week by the methods already described. On each occasion when a determination was made, twenty-four samples were taken from buoys anchored in the river at the discharge-site. The Public Health determination shows a close agreement with the average for the whole river. The latter records affords scope for an entirely independent line of research. The amount of silt in suspension not only varies from year to year, but the incidence of the peak of the curve is quite different from one year to another. The falling off of the peak may either mean that the amount of silt is reduced from the source, or else that the fall is due to the dilution of the mixture in the river by the addition of a supply of water less-heavily charged with silt. It would appear that the Atbara, which joins the Nile between Khartoum and Wadi-Halfa, has a considerable influence on this phenomenon.

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[1] W. B. Pollard, "Cairo Scientific Journal," vol. v, (1911), p. 139.

APPENDIX D.

Computation of the Beleida Formula.

By S. Lelyavsky.

Let it be supposed that the factor B in the general formula $V = BR^\alpha S^\beta$ is a variable, depending on the quantity of silt z expressed in grams per cubic metre. Then the formula becomes $V = f(z)\, R^\alpha S^\beta$, where

V denotes the mean velocity of flow in metres per second,
R denotes the hydraulic radius in metres, and
S denotes the sine of the angle of slope of the water-surface.

It is required to determine the value of B, using the data obtained in 1921 at Beleida. The process is as follows :—

The quantity of silt in suspension per cubic metre of water increases at the beginning of the flood and decreases as the flood subsides. Consequently there must be at least two dates on which the amount of silt in suspension will be the same. Let the hydraulic elements for the first day be distinguished by the symbols V_1, R_1, and S_1, and for the second day by V_2, R_2, and S_2; then

$$V_1 = f(z_1)\, R_1{}^\alpha\, S_1{}^\beta$$
$$V^1 = f(z_2)\, R_2{}^\alpha\, S_2{}^\beta$$

Now $f(z_1) = f(z_2)$ because, according to the initial assumption, $z_1 = z_2$. Consequently

$$\frac{V_1}{V_2} = \left(\frac{R_1}{R_2}\right)^\alpha \left(\frac{S_1}{S_2}\right)^\beta,$$

or

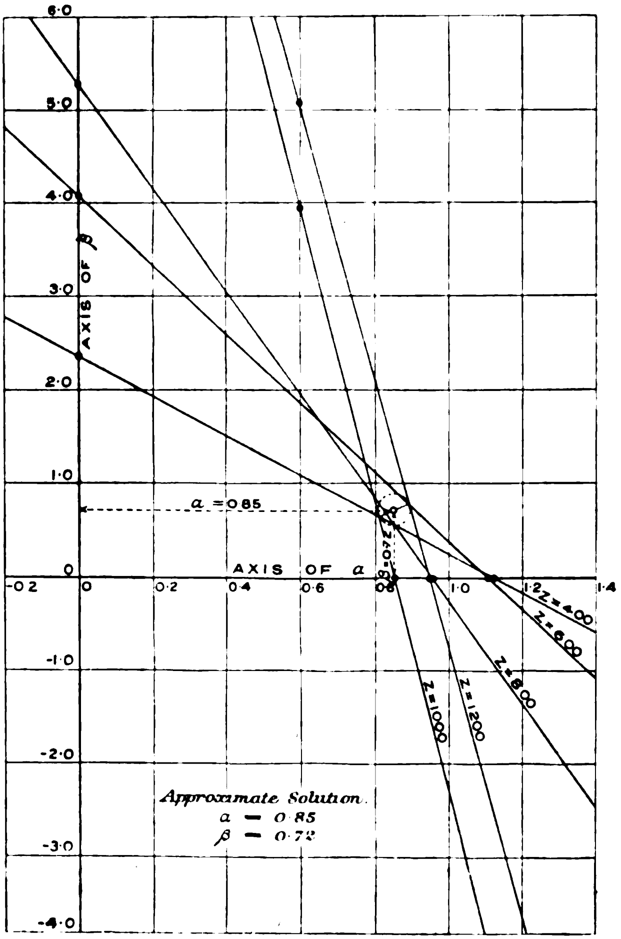
$$Log\left(\frac{V_1}{V_2}\right) = \alpha\ Log\left(\frac{R_1}{R_2}\right) + \beta\ Log\left(\frac{S_1}{S_2}\right) \quad . \ . \ . \ . \ . \quad (1)$$

This is an equation of the first degree for α and β, which yields a straight line when plotted graphically to axes of α and β.

This process may be applied for any 2 days having the same value of z. Thus a series of equations is obtained, each of which may be expressed graphically by means of a straight line. In the ideal case, where a general solution would exist, all these lines would intersect in one common point on the diagram. In practice, however, owing to accidental errors of observation, the curves fail to intersect in a single point. The first approximation may be taken as the centre of the smallest circle which can be drawn either tangentially to all the lines or as an intersection to them.

In the computation under review, the following values of z have been selected: 400, 600, 800, 1,000, and 1,200 grams of silt per cubic metre. Inasmuch as silt-determinations were only made once a week, values intermediate to those obtained by direct determination were obtained by interpolation from Fig. 10, Plate 4. Formula (1) was computed; the values are tabulated on p. 211, and are shown graphically by *Fig. 14*. The centre of the circle alluded to in the previous paragraph is located, and its co-ordinates represent the values of α and β, which are as follow:—

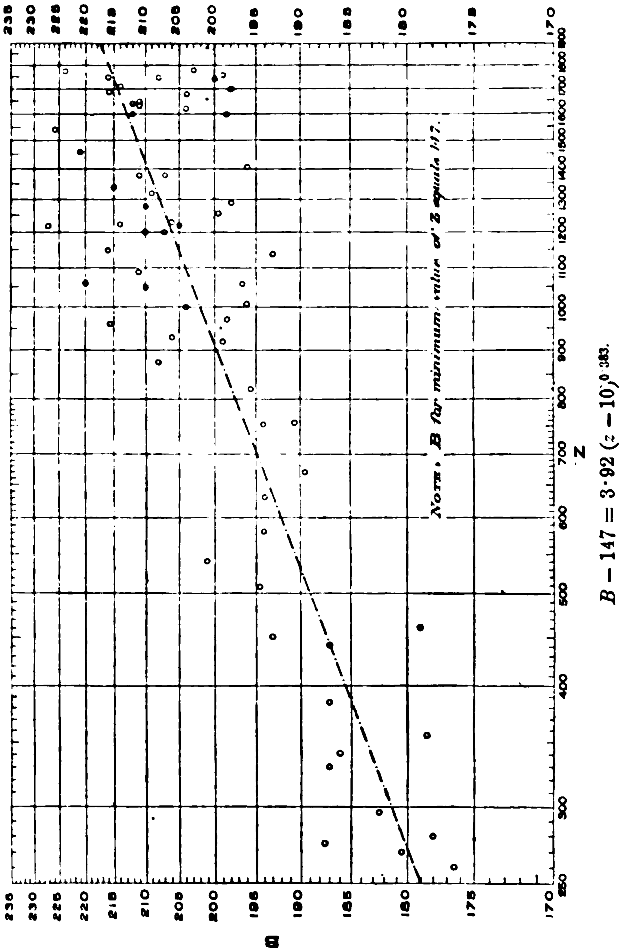
Fig. 14.



$$\alpha = 0{\cdot}85 \qquad\qquad \beta = 0{\cdot}72.$$

The next stage consisted in determining the values of B corresponding with these exponents for the observed values V, R, and S. For the low stage of the river B_0 equals 147, the silt-content at this period being approximately $z_0 = 10$ grams per cubic metre. The difference $B - B_0$ is then plotted on logarithmic paper against the difference $z - z_0$ (*Fig. 15*). The curve traced on this diagram represents the equation

Fig. 15.



$$(B - B_0) = 3{\cdot}92\,(z - z_0)^{0{\cdot}383}$$

Thus the formula becomes :—

$$V = \left\{147 + 3{\cdot}92\,(z - 10)^{0{\cdot}383}\right\} R^{0{\cdot}85}\, S^{0{\cdot}72}.$$

The velocities (Fig. 11, Plate 3) were computed according to this formula, using observed values of R, S, and silt.

It should be borne in mind that the determination of α and β have been based on a series of relations—

$$\frac{V_1}{V_2}, \frac{R_1}{R_2}, \frac{S_1}{S_2}$$

As the available observations are limited to a single year, the values of the numerators and denominators are generally of the same order, and hence the possibility of error in the result is aggravated by possible errors of observation. This difficulty can only be overcome by extending the same observations over several years. The amount of silt in suspension is not always the same from year to year, and consequently in other years considerable difference of V, R, and S may occur for the same amount of silt in suspension.

COMPUTATION OF EQUATIONS FOR *B* BASED ON OBSERVATIONS MADE AT BELEIDA IN 1921.

Z Silt. Grams per Cubic Metre.	Date.	*V* Mean Velocity. Metre per Second.	*R* Mean Depth. Metres.	*S* Slope.	$\frac{V_1}{V_2}$	Log. $\frac{V_1}{V_2}$	$\frac{R_1}{R_2}$	Log. $\frac{R_1}{R_2}$	$\frac{S_1}{S_2}$	Log. $\frac{S_1}{S_2}$	Equation.	Values of α and β. Used for Graphical Computation.	
400	(10–11)–8–21	0·65	4·30	0·0000685	0·650	−0·187	0·683	−0·166	0·833	−0·079	$187=166\alpha+79\beta$	$\alpha=0$	$\alpha=1{\cdot}126$
”	15–11–21	1·00	6·30	0·0000820								$\beta=2{\cdot}36$	$\beta=0$
600	(11–12)–8–21	0·75	4·75	0·0000725	0·625	−0·204	0·653	−0·185	0·892	−0·050	$204=185\alpha+50\beta$	$\alpha=0$	$\alpha=1{\cdot}11$
”	2–11–21	1·20	7·27	0·0000814								$\beta=4{\cdot}08$	$\beta=0$
800	(13–14)–8–21	0·915	5·35	0·0000785	0·678	−0·169	0·665	−0·177	0·930	−0·032	$169=177\alpha+32\beta$	$\alpha=0$	$\alpha=0{\cdot}955$
”	(22–23)–10–21	1·35	8·05	0·0000844								$\beta=5{\cdot}28$	$\beta=0$
1,000	15–8–21	0·979	5·72	0·0000785	0·715	−0·146	0·675	−0·171	0·976	−0·011	$146=171\alpha+11\beta$	$\alpha=0{\cdot}60$	$\alpha=0{\cdot}855$
”	7–10–21	1·369	8·475	0·0000804								$\beta=3{\cdot}94$	$\beta=0$
”	(15–16)–10–21												
1,200	17–8–21	1·060	6·06	0·0000792	0·776	−0·110	0·765	−0·116	0·982	−0·008	$110=116\alpha+8\beta$	$\alpha=0{\cdot}60$	$\alpha=0{\cdot}95$
”	27–9–21	1·368	7·92	0·0000806								$\beta=5{\cdot}05$	$\beta=0$

Discussion.

The President. The PRESIDENT moved a vote of thanks to the Authors.

Mr. Wilson. Mr. J. S. WILSON, on behalf of the Author, showed a series of lantern-slides illustrating Mr. Buckley's Paper. He thought this admirable Paper was one of the most valuable that had been read before the Institution; the information would certainly be of very great importance in connection with the hydraulics of rivers. Sir Murdoch Macdonald was adviser to the Ministry of Public Works in Egypt at the time when the experiments were instituted, and the thanks of the members were due to him for having sanctioned and aided them. The measurement of discharge in the Nile was not easy. That river had been surrounded by mystery since the earliest times, and for thousands of years measurement of its flow had been considered obviously impossible. At one time it had been thought that the river welled up from the rocks near Assuan, and Strabo described how early geographers had proved to their own satisfaction the correctness of this theory by lowering a plummet into the waters of the cataract, letting out all the available line, and failing to reach bottom. The Paper, of course, dealt in measurements, and in this connection he drew attention to Lord Kelvin's dictum that no progress could be made in any science except by measurement. Until a few years ago very little had been known about the flow of the Nile, but now with all the recent work, he did not suppose there was another river in the world about which so much information was available; and he thought that that contained in Mr. Buckley's Paper would result in great progress being made in the science of river hydraulics.

He had had one of the recorders brought to the meeting, with a view to explain its mechanism. The difficulty in all gear previously used was due to the fact that the transmitter and receiver or recorder at the distant end could get out of step. Where the float station was some miles from the recording station, it was not easy to synchronize the two. In the gear which was first installed by Mr. Buckley the transmitter and receiver could get "flustered" if the level of the water varied suddenly, the rapid rotation one way or the other of the drum carrying the tape suspending the float being too swift for the electric gear to keep pace with it; contacts were missed or electric impulses were sent in the wrong direction, Mr. Wilson.

and immediately the receiver gave faulty indications. In the present transmitter the connection between the float-drum and electric gear was through a differential, involving what might be called a repeater action. As soon as the float-wheel moved, a release allowed a contact to be made, and the current that would then flow through the recording circuit set back the release gear ready for any further movement of the float-drum. If the recording circuit was disconnected, or the current failed, the release was not set back, and the float-drum might even move through many revolutions without anything being recorded, but this movement would be stored up in the transmitter so that as soon as the circuit was completed the stored movement was transmitted to the receiver by the necessary number of electric impulses. In a similar way, no matter how rapidly the drum might be turned, the impulses would be sent to the recorder at the uniform rate, thus obviating the risk of missed signals.

Sir Murdoch Macdonald.

Sir MURDOCH MACDONALD, M.P., congratulated Mr. Buckley and Mr. Wilson on what they had done in helping forward the measurement of water flowing in open channels. He felt that he might refer with all the more reason to Mr. Buckley, inasmuch as he himself had had a little to do in the first instance with the initiation of the great advance in the measurement of rivers to which Mr. Wilson had referred. Years ago, when he took charge of the Assuan dam as Resident Engineer, he thought the water flowing through the sluices ought to be measured, and, in comparing what he had imagined to be the real discharge with that found by taking ordinary gauge-observations in the river itself, and further comparing these with a discharge-table, he found that there were very great divergences. At that time there was no particular reason to know the discharge in the flood season when the river was silty. To-day, as then, the important stage of the river was the low stage, from January to June. As a result of that initial work a great mass of investigation had since taken place. He was not satisfied with the coefficients obtained from observing sluice-discharges in the first instance, but he took advantage of a tank which had been originally built for the purpose of protecting a particular part of the rocky surface below the dam, as he had seen its possible utility as a measuring-instrument, and he had built the tank still higher so that it could contain a very large volume of water. With the information gained therefrom it was possible eventually to guarantee that the quantity of water passing down the river was correctly measured to, he believed, 1 per cent. In years to come the stage with which Mr. Buckley dealt—the silty stage—would be equally

Sir Murdoch Macdonald as important as the January–June stage; so that the investigation Mr. Buckley had made would prove distinctly advantageous if it would help to settle the principles on which measurements could be made easily and accurately at such seasons in the future. He was not sure that he could concur with all the conclusions Mr. Buckley had come to regarding the effect of silt in increasing the discharge of the river itself, and he believed the Scientific or Physical Department of the Egyptian Government, in whose hands in recent years a good deal of the work of measurement of the river had been placed, were also not of the same opinion as Mr. Buckley.

Mr. Buckley. Mr. R. B. Buckley remarked that the three Papers were of great interest to all irrigation-engineers. They referred to three different parts of the world, but they all dealt very largely with the same subject—silt. Silt was like many other things in this world; if it was treated well it was an excellent servant, but, if not, it was a curse to the irrigation-engineer. *Fig. 10* of Mr. Rothery's Paper (p. 173), which showed the monthly average of silt in the water of the Colorado river, was remarkable, inasmuch as the monthly average in one year was shown to be 22,400 grams per cubic metre. Obviously the maximum figure of that year would be considerably greater—probably between 30,000 and 40,000 grams per cubic metre. Some years ago he compiled information regarding a number of rivers, showing the maximum volume of silt carried by them. The Colorado river beat anything he had ever come across. Its silt-content was ten times the maximum of the Nile. The only other river he knew that even approached it in silt-capacity was a tidal creek in Lower Bengal, where it might almost be said that liquid mud ran down. It was therefore not surprising that the Colorado river produced some remarkable results. Mr. Rothery stated that the scouring of the river-bed in the flood was more than 20 feet in depth, which was a very high, though not an unprecedented figure; but it showed the very friable nature of the river-bed. It was well known that rivers which carried much silt in their deltaic regions raised their beds and banks, and this action in the Colorado river had caused the elevation of the bed by 13½ feet in 10 years, which, in turn, had necessitated increased height in the marginal embankments. Mr. Lacey's Paper also referred to this very old trouble in connection with marginal embankments. When a river was embanked, one of the objects was to protect a tract in the neighbourhood of the river, over which the water flowed and did damage. In many places protective embankments had been made, with the result that the strip of land between the river and the embankment, Mr Buckley.

which in some cases was some miles wide, was raised by the large quantities of silt which were deposited on it. Mr. Rothery mentioned a case where some contractor's wagons were buried to a depth of 4 feet at a distance of ½ mile from the river. An embankment made to protect a tract of land was often not so beneficial as was believed. He knew of several cases in India where the value of the land which was protected was considerably less than the value of the land not protected, although the crops on the unprotected land were frequently destroyed by the floods. The benefits resulting from the deposit of silt were so great, that it was more economical to lose the crop occasionally than to get a poorer crop every year on protected land. In India the Gunduk river flowed down from the north and joined the Ganges near Patna. Both rivers spilt, the Ganges spilling over its left bank and the Gunduk over its right bank, and the floods very nearly met at certain points. The natives said that in a year when the spills from both rivers were large it was possible to trace plainly, in the cold weather, the line of meeting of the two floods. The Ganges river contained silt of greater fertilizing property than the Gunduk, and it was only necessary to observe the good crops on one side, and the indifferent crops on the other, to discover where the floods met. Mr. Rothery paid a tribute to Mr. R. G. Kennedy, and mentioned that a great deal of trouble ensued in the Colorado irrigation-canals because of the deposits in them. Mr. Kennedy made various experiments and arrived at his "critical velocity" formula, the value of which was not yet fully appreciated by irrigation-engineers. When that formula was used, with certain corrections applicable to particular circumstances, it was possible, in nearly all cases, to design channels which would not silt. Mr. Rothery regretted that that formula had been neglected in the design of the Colorado channels, and Mr. Buckley was glad to see that he had brought that important point forward. Mr. Rothery referred to an astonishing proposal to build a masonry dam 500 feet high. Mr. Buckley did not understand why the weir shown in Figs. 12, Plate 2, had not been breached. It was situated in a river with a very friable bed, and he would have thought that, as soon as the level of the river was raised above the weir, the water flowing over the top of the weir would cause erosion below the weir and lead to a breach in it. Mr. Rothery's description of the method of construction of the weir was very interesting. It was, Mr. Buckley believed, entirely novel. Mr. Lacey referred to the silting-up of the heads of canals. Appendix C of Mr. A. B. Buckley's Paper indicated the cure for that

Mr. Buckley. trouble. It was a very old story in India, and he thought the difficulty had been quite overcome. Mr. Lacey said that, when a weir was constructed across a river, the bed of the river was more or less raised, sometimes perhaps up to the very crest of the weir. When a canal was constructed, the floor-level of the head-regulator was generally put some feet above the level of the bed of the river, and, as the bed of the river rose, the height of the floor of the head-regulator above the river-bed became less. It was well known that rivers carried down silt in suspension, but he did not think it was always realized that the flow of sand and pebbles along the bed itself was large. As was pointed out by Mr. Lacey, if the bed sand of the river was allowed to run into the canal, which generally had a much lower velocity than the river had, the sand, when carried into the canal, stayed there and obstructed the flow. One canal he had had to do with was dredged out every year for 20 years or so until a simple remedy, discovered in the first instance in the Punjab, was applied. In most regulators the sluices opened from the bottom, and consequently, when the water-level of the river was considerably above the level of the canal, the high velocity underneath the sluice carried the silt travelling along the river-bed into the canal and choked it. He had known one particular case where, if a shutter was left down 3 or 4 feet above the floor of the sluice, the bed of the river, flowing up and over the shutter into the canal, actually carried stones the size of a man's thumb over the top, simply due to the velocity. If, instead of the vents of the head-regulator being opened from the bottom, they were opened from the top, the bed of the river was not scoured into the canal. It was an almost infallible cure, but might not always be financially possible. The ventage into the canal had only to be at a high level, and of sufficient capacity to carry the required discharge, and the bed of the river would not flow into the canal and choke it. It might be necessary to extend that narrow opening to a considerable distance to limit the velocity. He considered that the Buckley-Wilson recorder was one of the most important inventions that had ever been laid before irrigation-engineers. It was an instrument which, if it could be easily utilized in the various channels of an irrigation-system, would be of the utmost value in regulating the discharges. Mr. A. B. Buckley's theory of the effect of silt on the velocity of water flowing in open channels was, he considered, fully established by his Paper, but it was necessary to study that Paper carefully, with its numerous diagrams, to appreciate fully the force of the arguments. For his part he was convinced that the theory was sound, Mr. Buckley. and he hoped that the experiments and observations in Egypt would be continued, so that the theory might be still further investigated. Briefly, the theory put forward was that silt-bearing water lubricated by its deposits the periphery of a channel and reduced the friction between the water and the surface over which the stream flowed. There was, however, another factor bearing on the effects of silt on canal-discharges. When clear water flowed down a canal, light was able to get to the bottom; when silty water flowed, the light was obstructed. Weeds grew quickly in clear water, but, as soon as silty water shut out the light, the weeds disappeared in a marvellous way. He had seen canals, running normally with a slope of about 6 inches to the mile, in which weeds had grown to such an extent that the surface slope was two or two and a half times the normal. Weeds were an important factor in reducing the discharge of a canal, and in many cases it was the silt which killed them.

Dr. Unwin. Dr. W. C. Unwin, Past President, said it would be recognized that a very striking and original Paper had been presented by Mr. Buckley, who had come to the rather unexpected conclusion that, at a given gauge-level of the river, the velocity was greater when the water was heavily silt-laden than when it was clear. So far as he could judge, Mr. Buckley had made out his point, though probably a good deal of discussion was wanted as to what exactly that result meant. The first point to observe was the exceedingly ingenious recorder which had been applied at one station in Egypt. It was simple in principle, although it might be difficult to construct. There was only one possible objection he could see to it. Its functioning depended on measurement of slope, and the slope of the river was taken between a point 2 kilometres above the gauging-station and another 3 kilometres below it. If the stream was what the older hydraulic engineers always assumed—a stream with a permanent bed, and of uniform section and slope, with the water-surface parallel to the bed—it would not matter where the measurements were taken; but the Egyptian streams with sandy beds seemed to him to comply rather badly with those conditions, and there was just the chance that the slope measured in 5 kilometres might not be the slope exactly at the gauging-station. There were indications in the Paper —although he could not satisfy himself exactly about it—that at times the slope of the stream was not the same as the slope of the bed. That could not continue for 100 miles, but it might continue over a considerable section of the river, and if there was a difference between the slope of the surface of the stream and the slope of the bed, then the equations which Mr. Buckley quoted so often—and

Dr. Unwin. sometimes a little abused—were not the equations properly applicable. It was a case, not of uniform motion, but of varied motion, which had been more or less investigated. He did not know that any attempt had been made to apply the equations of varied motion to the solution of problems of the kind dealt with in the Paper, but possibly they might explain what was rather puzzling, namely, that the discharge in a rising condition of the river was different from the discharge in the falling condition of the river, for the same gauge-level. The whole prosperity of Egypt depended on the supply of irrigation water. It was of enormous importance, even of political importance, and of importance to this country. Sometimes the supply fell a little short of the whole requirements, and certain of the less-valuable crops had to be starved to keep up the supply to the others. There was hope that in the future this country might obtain from Egypt and the Sudan, with sufficient irrigation, a million bales of the best cotton annually; and it would be a happy thing for that supply to come when the supply from America was decreasing on account of the Americans requiring their own cotton for their own use. There had been built up in Egypt a Public Works Department and a Physical Department in which there were staff officers of the very highest scientific attainments, who during late years had carried on hydraulic researches which were admirable. The importance of the question of the measurement of water in Egypt was so great that they were able to carry out researches on a scale and at an expenditure which he believed was impossible anywhere else. Mr. Buckley explained, quite rightly, on p. 186 what he meant by the friction of water. Dr. Unwin was sorry that the word friction was ever used in that case, because it was wholly different from what was ordinarily understood by friction. Mr. Buckley had traced the fluid friction to the creation of eddies at the bed of the stream which absorbed part of the energy of flow, an energy which was afterwards dissipated and rendered useless by the viscous resistance of the water to shearing. Mr. Buckley's theory was that in heavily silt-laden water a blanket of slurry formed at the bottom of the stream over which the rest of the water slid with a less production of eddies than would otherwise be the case. That was a possible theory, but it was rather difficult for one of the older hydraulicians, knowing something about the turbulence of water, to realize that such action could occur. That was definitely Mr. Buckley's theory, but he sometimes departed from it. On p. 187 it was said that the presence of silt in suspension in the water did not necessarily alter the coefficient of friction, and on p. 197 Mr. Buckley said that it appeared to be the consistency of the slurry at the bottom which influenced the friction and not the amount of silt held in suspension. Then, at one of his principal gauging-stations he had deduced a formula of flow which was intended to take account of the presence of silt in the water—the Beleida formula. That formula had a coefficient which depended entirely on the number of grams per cubic metre of silt in suspension in the water. It did seem to fit with very great accuracy some of the cases the Author had examined, but it did not seem to be quite consistent with Mr. Buckley's theory of the action of silt in the water. Mr. Buckley quoted some of the well-known formulas for the flow of water in streams and canals and said that they were all unsatisfactory. All those formulas were devised for a uniform flow of water in permanent channels. He was afraid practical engineers were rather too fond of getting hold of a formula—it saved them a good deal of trouble—and applying it in conditions to which it was never meant to apply, and then abusing it because it did not answer. It was quite true that the formulas did not answer in the cases which Mr. Buckley was chiefly considering, but the reason was that they were never meant to apply quite to those conditions. In considering the Paper it would have helped very much if Mr. Buckley had given some vertical velocity curves for a given gauge-level in clear and in silted water. Such curves would have thrown light on the question of the blanket of slurry and its velocity. As he had criticized the Paper, he would like to say that he had not criticized it because he did not recognize that it was very important, but only because the subject required very full discussion. Dr. Unwin.

Professor S. M. DIXON remarked that the importance of the transportation of silt in rivers and water-channels was being recognized more and more by hydraulic engineers every day, and the observations that had been carried out by Mr. Buckley showed the necessity of a knowledge of the silt in a river when it was desired to estimate the discharge. The results obtained were of the utmost consequence, and, as Dr. Unwin had said, they could only have been carried out where the work had been well thought out by the Departments concerned. It had cost a large amount of money, and had been carried out by a number of highly-skilled observers. Although the observations and the results obtained were so important, the explanation given by Mr. Buckley did not appeal to him. Mr. Buckley said on p. 186: "It appears therefore that the silt has a lubricating effect," and he then proceeded to explain how that worked. Was it necessary to assume that the water Professor Dixon.

Professor Dixon.

which was carrying the silt acted like a heavier liquid and damped out eddies and whirls formed in hollows and inequalities of the river-bed? Mr. Buckley's assumption did explain what happened, but it was known, and Mr. Buckley had proved, that silt which was being carried down by the river was deposited in the bed, in those inequalities, and so probably in a way prevented the eddies from coming up; in fact it was deposited and formed the smooth surface to which Mr. Rothery referred when he said that the silt that came down formed a smooth deposit, which was a factor in increasing the discharge. The advantage of that hypothesis was that it also explained exactly what occurred when a river was in flood and was not fully charged with silt, as frequently happened. The first water that came down was fully charged with silt. Afterwards the silt might be deposited or carried further on, and the water was no longer charged. As more water came down, the silt was eroded from the bottom of the river, and inequalities were formed by the current: in that case the velocities would be reduced. With regard to the determination of the hydraulic radius, Mr. Buckley, he was sure, would agree that it was a very much more difficult problem than would appear merely from reading the Paper. Measurements made when soundings were being taken in a river which was moving fast and which was deep, were very laborious to obtain and were located with difficulty. Those referred to in Mr. Buckley's Paper were located by box-sextant observations, which did not seem a very exact method. In making measurements in Canada, when surveying a river, through holes in the ice along a base measured on the ice, he had found the utmost difficulty, when the river was deep and swift, in getting consistent observations of the soundings taken through the holes, and three or four observations were necessary to get a fair average. In the case of a river like the Nile, and where the soundings were only taken at 50-metre intervals, in a high current and with a rapidly changing bottom, he thought the hydraulic radius so determined must be really a rough approximation. It was a very difficult and important quantity to determine. The only recent experiments that he knew of as worth recording, in connection with the transportation of silt in water, were those of Mr. G. K. Gilbert.[1] Mr. Gilbert's observations, instead of showing that silt in running water tended to increase the velocity, showed really that there was an opposite tendency—that the presence of silt would reduce the velocity. His conclusions were the following :— Professor Dixon.

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[1] "The Transportation of Débris by Running Water." U. S. Geological Survey: Professional Paper No. 86, p. 228 *et seq.* Washington, 1914.

> The available data are not fully demonstrative, but they render it highly probable that, under all conditions, streams are retarded by their suspended loads. . . . The average of the thirteen velocities found for loaded streams was 10 per cent. less than the corresponding average for unloaded streams . . . it appears that the reduction of mean velocity by traction is greater for small loads than for large, for gentle slope than for steep, for low velocities than for high, and for large depths than for small . . . the cause of this is possibly connected with a fact brought out in . . . the discussion of vertical velocity curves—the fact that addition of load has a pronounced influence on the distribution of velocities, increasing the contrast between velocities near the bed and the mean velocity. (A 10 per cent. average reduction of velocity corresponds to a 0·52 per cent. by weight tractional load.) . . . In connection with the vertical velocity curves, the conspicuous change associated with the addition of load is the raising of the level of maximum velocity.

In Fig. 4, Plate 3, Mr. Buckley showed the velocity at 1 metre from the bottom in the case of the silt-laden river and the river without silt. It would be very interesting if Mr. Buckley would supplement that with the vertical curves of velocity, since, as by stated Mr. Gilbert, they should show important differences: in that way further light might be thrown on an interesting subject.

Sir Thomas Ward.

Sir THOMAS WARD congratulated the Authors on the very interesting studies which they had put forward. The Papers perhaps did not contain work which would be used at once on the drawing-table, but engineers who had to deal with large hydraulic problems would like to refer to them in order to clear their ideas. The time available in a busy engineer's life for making measurements to put before The Institution was very limited, and the members would appreciate the good fellowship of the Authors in bringing forward their work. There was a story current in the Punjab that the late Mr. R. G. Kennedy was once surprised in a brown study on a canal-bank, and, on being asked what he was thinking about, replied : " Silt." Mr. Kennedy tested his ideas by measurement, and his Punjab Irrigation Branch Paper No. 10 was one of the classics of hydraulic engineering, not because it was a finished scientific study, but because of the exceedingly practical way in which Mr. Kennedy dealt with the problem before him. He had to remodel a very large canal, which, in the course of years, had outgrown itself, and engineers were discussing the principles on which the work should be done. In a very short time Mr. Kennedy gave coefficients sufficient to guide the work, and Punjab

Sir Thomas Ward.

canals were still designed more or less on the lines he then laid down. The young "Kennedys" of the present day, working in India and Egypt, knew that his results were quite empirical, and they were still studying the subject in order, if possible, to elucidate the underlying theory. Those who were working at it might include in their measurements estuarial channels, which had a cross section of constant mean velocity. Mr. Rothery's Paper gave figures for population and area irrigated, which indicated one person to 10 acres of land. In countries such as Sir Thomas Ward had had to deal with, such as Mesopotamia, the Punjab, and Seistan, the figure was more like one per acre. That figure included everybody who lived in the irrigated tract. Mr. Rothery spoke of the pitching on the levees attracting the river. In India, when it was required to push the river away, quick-growing bushes and coarse grasses were planted; but if it was desired that the river should follow a training embankment, stone was used. With regard to the trouble with rats, in India it had been found useful to put up perches for hawks, or to grow trees, where they would not be injurious to the embankment. In general, questions connected with the hydraulics of rivers could be considered in two main parts, those of the catchment, where all the watercourses on the map were drains bringing down the rainfall and detritus, and those of the delta, where the water was moving that detritus into a basin or the ocean. Many rivers had a long channel connecting the two, in which there might be the phenomena of the catchment or of the delta, as in Kashmir. The curve of the bed slopes of the river, according to Dr. Unwin's "Hydromechanics" tended to be parabolic, and that was an important consideration in taking off canals from rivers and in design. Usually the bed slopes were fixed by the slope of the ground, and perhaps without much thinking engineers naturally adjusted them to suit the character of the silt in suspension. In a delta the water was distributed and provided a large number of natural canals; in fact, the irrigation systems of Mesopotamia and Seistan—and probably Egypt—grew up in that way. When the main river deserted any part of its delta the people were able to control those effluents and to convert them gradually into canals. Effluents took out over a natural bar. A study of the bar showed that it was formed where the stream-lines broke. The current deposited its silt and gradually built up a bar, which was scoured down when the clear water came if the supply in the river was maintained. Usually the river dropped, and the canal ran partially dry. In ancient times, as the people became more civilized, they made rough-and-ready weirs of fascines, and so kept the canal going for the benefit of the crops in the low-water season. The stream-line flow at the mouth of such an effluent was of great interest to canal-designers. It showed the way in which Nature selected the currents in the parent channel for the service of the effluent, and that accounted for the fact that those channels seldom or never silted up solid, but kept going from generation to generation. Sir Thomas Ward.

Mr. Elsden. Mr. F. V. ELSDEN remarked that Mr. Rothery's Paper was of particular interest to him, because many of the conditions which Mr. Rothery described closely resembled those prevailing on irrigation works in the Punjab, where he had been engaged for the past 18 years. It would be interesting to know what was the most common cause of the numerous breaches in the levees of the flood-prevention works, to which Mr. Rothery referred on p. 169. The extremely rapid rate at which the river-bed had risen suggested that they were sometimes due to the flood-waters overtopping the levees, though perhaps they were more commonly due to infiltration water scouring out voids in the bank or in the soil beneath it, as mentioned on p. 169. He noticed, however, that it had been found necessary to revet portions of the levees to resist direct scour. The revetment (*Figs. 7* and *9*, p. 170) appeared to lack what in Punjab practice was considered essential, namely, a flexible apron or berm of loose stone, supporting the toe of the revetment, without which breaches might not infrequently result from the flood-waters scouring out the bed of the channel at the toe of the revetment. In India, a levee provided with a flexible apron of stone (a " Bell bund ") was found to be almost invulnerable, provided the apron was properly proportioned and maintained. The novel type of weir employed was full of interest, and he would like to have further details of the working of the mattresses; e.g., some information regarding their removal would be welcome: the silt deposit which occurred within and upstream of the mattresses might render that removal difficult. Had any trouble been experienced in that respect, and was it possible to use the mattresses for more than one season, or had new mattresses to be made every year? In the latter case there would be considerable recurring expenditure on maintenance. With regard to the reason given for not adopting a permanent weir, he called Mr. Rothery's attention to a Paper[1] by Mr. F. H. Burkitt, in which it was demonstrated that for every depth of water there was a certain limit of height for a weir which would cause no afflux. It would seem

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[1] "Experiments on Broad Crested Weirs." Proceedings of the Punjab Engineering Congress, 1919, p. 103.

Mr. Elsden. that it might be possible to construct a permanent weir sufficiently high to be economically practicable, and at the same time without incurring any risk of flooding lands upstream of the weir. The maximum river-discharge of that project was almost identical with that of the river Ravi at Balloke, the head of the Lower Bari Doab canal, where the diversion-weir consisted of a barrage of thirty-five bays of Stoney gates, each of 40 feet span. That length of waterway, however, was considerably more than was really necessary, and a cheaper and more satisfactory barrage would have resulted from a reduction, combined with a lower crest-level for the weir. At Balloke, as in the Californian project, the offtake canal had a capacity of 6,000 cubic feet per second, which, since the Lower Bari Doab canal was financially successful, suggested that the prime cost of a permanent weir on those lines might not be excessive in California. It would certainly result in simplification of working and reduction of maintenance-costs, and it would also give at all times better control of the river, and make it possible to maintain a more stable bed in the neighbourhood of the diversion-works. Rockwood gate was remarkable by reason of the length of waterway provided, which was more than double that which would ordinarily be considered sufficient for such a canal in Punjab practice. The use of flashboards to form a raised sill, and presumably, also, for regulation of supply, would also not be considered satisfactory in the Punjab, where flashboards had long been given up on all works of any size in favour of a permanent, raised masonry sill in combination with a rising-sill gate operated by standing gear. The great length of waterway of the gate in question probably arose from a desire to exclude silt from the canal; but the serious difficulties in connection with silt which Mr. Rothery described did not indicate that it had been of much advantage, and, indeed, the efficiency of flashboards or any other form of raised sill for that purpose was often seriously overestimated. It was stated that only the upper portion of the water was taken. That was a correct statement of the generally-accepted view of the action of a raised sill, but nevertheless, in his opinion, it did not represent what actually occurred where such a raised sill was employed. He had shown in a Paper[1] published in 1922 by the Punjab Government that the use of a raised sill, such as was formed by the flashboards in the Rockwood gate, did not result in the upper portion alone of the water being taken into the canal; on the contrary, water from the whole depth of the river was admitted, the only effect of the raised sill being to cause the whole of the water which entered the canal to rise over it. Mr. Elsden. By thus causing the water to rise over a high sill, some part of its silt-content was probably left behind in the river; but that process was certainly incomplete, and there could be no doubt that a great deal of silt from river-levels lower than the level of the sill was swept over by the rising water. The extent to which action of that nature might be harmful was shown by the analyses of Nile silt given in the Table in Appendix C of Mr. Buckley's Paper. A raised sill was a device of very small value, except in regard to rolling silt, unless it was made exceedingly long, so that the upward velocity of the water rising over it was negligible. In order to improve silt-separation at canal-heads he had developed a type of head regulator and gate (described in the Paper already referred to) which skimmed off the top layer of water, leaving behind in the river both the water and silt of the lower layers. That type of regulator could be designed to exclude all but the upper portion of the three layers of water referred to by Mr. Lacey on p. 158. The method consisted merely in superimposing the orifice of the canal headgate over a lower orifice, which was separated from it by a horizontal partition, the two orifices being so proportioned that the upper one drew water from above the level of the sill, all water from below the level of the sill being passed into the lower orifice and thence back again into the river. In order to utilize to greater advantage the velocity of approach of the water in the river, the plane of the orifices was placed preferably in a position transverse to the direction of flow of the river; and, in order to adjust the sill-level to the varying stages of the river, a flap gate was hinged along its downstream edge to the permanent sill of the canal-gate orifice, so that by rotating it about the hinge, its upstream edge could be adjusted to different levels. Although the prime cost of such a gate for canals having a capacity of more than about 500 cubic feet per second would be greater than that of the old type, the excess would very soon be recouped by reduction in cost of maintenance. The project described by Mr. Rothery seemed to him to be essentially one in which advantage would be obtained from the adoption of a head-regulator and gate of the type referred to. On many Indian canals silt was a source of serious trouble, and the additional cost of such a head regulator would be well justified; but on none of them, so far as he was aware, was it now necessary to resort to the constant dredging required on the project which Mr. Rothery described. It was of interest to note that on that project, as in the case of many Indian canals, trouble had been caused by rise of the water-table and by waterlogging of the soil, accompanied by a rise

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[1] "Irrigation Canal Headworks." Punjab Irrigation Branch Papers, No. 25. Lahore, 1922.

Mr. Elsden. of alkali to the surface. That was a problem which, of recent years, had become very prominent in the Punjab, and for which no satisfactory cure had yet been found. In his own opinion that condition was due almost entirely to percolation from the canals, and the only effective cure would be the application of a waterproof lining to canals. While both cheap linings and efficient linings could be made, a lining combining both cheapness and efficiency did not seem yet to be available. He had suggested several years ago that the solution might perhaps be found in some form of bituminous sheeting, and extensive tests of such material were now being carried out in the Punjab. Deep drainage, combined, probably, with underdrainage, would almost certainly obviate the evil effects of the rise of the water-table, but would be very costly, and at best only a palliative; whereas a suitable lining-material would strike at the very root of the trouble. It seemed to him hardly a matter for surprise that the vented dams described by Mr. Lacey had not proved very effective in reducing the quantity of silt entering canals. Opinion in the Punjab now favoured a barrage, having its crest at as low a level as possible, and its linear waterway as small as possible, with the object of constricting the river, so as to force it into a deep, narrow channel rather than a wide, shallow one. That would greatly facilitate the maintenance of a clean bed at low stages of the river and eliminate deep local scour. The principle had been largely applied to bridges by railway-engineers in India with highly satisfactory results, and was now being adopted on a large new canal project in the Punjab. The most striking example with which he was acquainted of its application to bridges was the Alexandra bridge over the River Chenab at Wazirabad, which was originally constructed with sixty-four spans and had now been reduced to seventeen spans, each of the same width, with very satisfactory results. That principle, of course, called for proper training of the river both upstream and downstream of the constriction, were it bridge or barrage; but it would, even so, generally lead not only to better control of the river but also to great saving in outlay. If, in combination with a barrage of that description, a head regulator of the skimming type were adopted, the result should be, in all cases, a solution of the silt problem, and in some cases a reduction in cost, in spite of the higher cost of the regulator. There would still be, no doubt, variation in the silt-content of the water admitted to the canal, but both the range of variation and the maximum silt-content would be reduced to an extent which would give little, if any, trouble in the canal-system. That solution would, he thought, be both cheaper and more satisfactory than the curved weir and scouring-sluice suggested by Mr. Lacey. Mr. Buckley's Paper marked a very real advance in knowledge of the effect of silt on the flow of water. Some observations on that subject came to his own notice a short time ago in the Punjab, in which the same reduction of resistance to flow was noticed during periods when the water was heavily laden with silt; but no sufficient information was obtained to render possible any attempt to explain the phenomenon. Those observations were made on a length of canal having a firm bed of good soil, and no indication of either silting or scouring was reported, but that left the possibility of there being temporary precipitation of a part of the silt in the form of a slurry flowing along the bed, as suggested by Mr. Buckley. The observations related to a purely temporary condition of high silt-content, the duration of which was perhaps too short for any part of such a silt deposit to solidify, so that no trace of any precipitation would remain after clear-water conditions again became established. The analysis of the silt-content of Nile water, given in Appendix C (p. 204), was very interesting, and showed clearly how necessary it was, if silt trouble was to be obviated, that the upper layers only of the water should be admitted to a canal, and that anything which might tend to cause a rise of silt from the lower layers to the upper layers should be avoided. At the same time, it also showed that the exclusion from a canal of the lower layers of water would not result in any marked diminution in the quantity of useful fertilizing silt, classified in the Table as "fine silt and clay," which was taken into the canal, so that the rigid selection of the top water for admission to the canal would exclude only that part of the silt which was harmful. Mr. Elsden.

Mr. J. S. Beresford asked, with regard to Mr. Rothery's Paper, how the temporary mattress weir, which caused a heading up of several feet, could be constructed without protection against scour downstream. He had had, on the Ganges, a good deal of experience in closing branches of the river in sand, but his last experience was in closing, in April, 1905, two large and deep branches of the Nile, where the heading up was not nearly so great as on the Colorado weir. In order to protect the bed from scour, he repeated what he had done in India on the Ganges. Two carpets of strong sail-cloth were employed, each 66 feet wide, one 330 feet long, and the other 490 feet. A heavy chain was bound in a wide hem on the upstream edge, providing a strong base-line with rope attachment back at intervals of 25 feet to anchors. The carpet was laid across each channel from a bridge of boats, and fully protected the Mr. Beresford.

Mr. Beresford. sandy bed from scour. With regard to Mr. Buckley's Paper, the first thing that struck him was that there were a good many inconsistencies, such as Dr. Unwin had pointed out. Reading distant marginal gauges simultaneously to millimetres seemed to be an undue refinement in connection with a river which was very broad and subject to the north wind, and where the other hydraulic elements were measured far less accurately. In India, where he had had much experience of measuring river-discharges, the difficulty had been sometimes to get within 10 per cent. of the correct cross section of the channel. In large canals also that was once a difficulty; but it was overcome by building discharge sites 200 feet long, which enabled the cross section to be measured accurately; and mean velocities were taken by the method of loaded rods, which had always been found reliable. He had never looked to get a higher discharge with silt-laden water than with clear water; it was always thought in India that silt-laden water was further removed from the condition of a perfect fluid than was clear water, and that that would reduce the velocity to some extent. He had seen slurry in Egypt as dredged out of the canals, and in that semi-fluid state it would not stand at any slope. He had experimented with the finest sand he could get, under water, and found there was nothing of the nature of slurry about it; it took the same natural slope as in air. Therefore he thought that the theory of slurry making a smooth surface for the water to glide over would not hold good in practice. With regard to the blanket of silt, the grains that dropped would be only the larger ones, because, with the velocities obtaining, all the small and colloidal matter would be kept in suspension. The analysis given did not strike him as being of the kind that was required; the size of the grains of coarse sand in the Table on p. 206 was not stated, and much depended on size in connection with suspension. He had experimented with fine sand (passed through a 90-mesh sieve), and the velocity with which it fell through still water in a vertical glass column was about 1 inch per second. Hence the upward component of very small eddying motion would hold such sand in suspension. Consequently silt passing through a 100-mesh sieve would not drop where the mean velocities of flow, as described, were nearly 1·7 metre per second. If the peripheral velocity increased when the silt had fallen, it was hard to understand the cycle, for the figures on p. 189 and the Author's comments thereon showed that the increased velocity itself washed away the blanket of slurry which was assumed to have given it birth. It would seem inconsistent to have the velocity at the bottom increased and yet

Mr. Beresford. have a lighter deposit remaining on the bed; also, it was generally accepted that in every river sand was in constant movement at the bottom. He agreed with Professor Dixon as to the uncertainty of getting the correct cross section of the river from soundings 50 metres apart and only in one line. In India three sections were usually taken, and the average was adopted. Even when sounding from a boat in moderate depths with a long rod there was some difficulty in getting correct depths. With regard to the prevention of silt entering a canal, which had always been a source of great anxiety in India and in Egypt, he agreed with Mr. Elsden that a raised crest had not the effect expected. A very slight vertical velocity took up ordinary silt and carried it over the raised crest. A good example of that was described in Mr. R. B. Buckley's well-known work on Indian irrigation,[1] in the case of the Sutlej canal. It was thought at one time that the canal would have to be closed during half the year on account of the silt carried into the canal from the River Sutlej during the flood season; and as a remedy the crest of the headwork was raised 7 feet above the floor, with provision for raising another 3 feet by a movable gate behind. But the silt went over as much as before, and the final remedy was to build a divide-wall upstream from the weir end at the sluices which admitted of the controlling of the velocity past the head of the canal. By that means the larger portion of the silt was caught in the river-channel above, and at certain times the canal was closed and the collected silt was flushed out through the sluices. The hydraulic elements for the Khannaq site on the 4th September, 1920, were:—discharge 8,100 cubic metres per second, wetted perimeter 565 metres, slope 1 in 13,666, mean velocity 1·53 metre per second. Calculating with those figures, it was found that the frictional resistance of the wetted surface was 1·08 kilogram per square metre, or 0·22 lb. per square foot. Experiments by Mr. Froude showed that with a velocity of 10 feet (3·05 metres) per second on boards 2 feet long, covered with fine sand and coarse sand, resistances of 0·81 lb. and 1·1 lb. per square foot respectively were obtained. He had seen an account of Mr. Elsden's plan of skimming river water for canal supplies in *The Engineer* of November, 1922. In certain canals he thought it would work all right, but it would be very difficult to apply it to deep canals in Egypt, where the difference of level was so great, or on the proposed Sukkur canal on the Indus in Sind.

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[1] "The Irrigation Works of India," p. 38. London, 1905.

Sir Murdoch Macdonald.

Sir MURDOCH MACDONALD, M.P., desired to add a few observations on the Papers by Messrs. Lacey and Rothery. Mr. Lacey said: "This force is due to the rotation of the earth causing any moving body on the earth's surface to be deflected to the right in the northern hemisphere and to the left in the southern hemisphere. That such action takes place is apparent from the tendency of rivers in the northern hemisphere, especially those flowing in a north and south direction, to erode their right margins." The course of the Nile lay almost due north and south; but he did not think anyone could say that such action took place as far as the Nile was concerned, and certainly it would not be reasonable to base anything upon that deduction. The Nile wandered from side to side, and apparently had done so from time immemorial. Mr. Lacey's Paper was very interesting, and he had been glad to have the pleasure of reading it. With reference to Mr. Rothery's Paper, he was sorry the Author had not shown the water-level in *Figs. 7* and *9* (p. 170), nor did he mention in the text to what height the banks had been built above water. One of the sections showed reinforcement on the land side. In Egypt, in more recent years, 1 in 7 had been adopted as the water gradient, drawn from the maximum water-level through the bank to the toe on the other side. None of the banks illustrated in the Paper would comply with that condition. He noticed that the materials of which the banks had been formed were as bad as, if not worse than, those in Egypt. He was astonished to see that 120,000 acres had been too much damaged to permit of economic production of crops. In all the schemes he had known allowance had been made for ultimate drainage. There seemed to have been a deep enough hollow into which water could have flowed, and he could not understand why provision had not been made for subsoil drainage, as was done in Egypt, and as had been arranged for in the big scheme now in hand in the Sudan. It had been fully realized there that, though drainage would be unnecessary for possibly 10 or 15 years, the time would come when it would be essential, and the plans were therefore prepared in such a way that a drainage scheme could be interlaced with the irrigation scheme.

Mr. Western.

Mr. R. W. WESTERN considered that Mr. Buckley's inference that, other conditions being the same, the discharge of the Nile was augmented when silt was present, was very important. He believed it was new, but it was just what he would expect. When clear water was flowing through a channel such as the Nile, or an irrigation channel, it overcame certain frictional resistances, some of which were undoubtedly proportional to the discharge in cubic feet per second. No one would contend that those resistances were increased by the presence of the kind of silt which was maintained wholly in suspension. If the discharge remained the same and the silt were added, the energy available for overcoming the resistances which was measured by the mass and the fall in level, was increased, whereas the resistances depended on the discharge in cubic feet. The increase might be equivalent to a fall of several inches in a mile, which would suffice to account for the increased flow observed and recorded in the Paper. There was another kind of silt, namely, that which was just on the point of being deposited or picked up. A stream running clear had frictional resistance to overcome, much of which was due to eddies caused by interruption of stream-lines, which were the effect of variation in the cross section of the channel. It was not unreasonable to say that, where the velocity of the stream was high, that kind of silt would be swept away, and where it was low, it would be deposited, so that it would be deposited where the cross section was large and swept clear where the cross section was small, thereby tending to make the channel more uniform and more efficient. Where there was an enlargement of a channel, it could often be seen that the surface of the stream became convex, and therefore the bottom of the stream was under a varying pressure, greater at the middle than at the edges; so that the bottom of the stream, which was being retarded more than the surface, was under a differential pressure which tended to push it to the sides of the stream and deposit there the silt swept clear from the narrow section; that tended to produce a more efficient channel, in which the water was transported with less loss of energy. It seemed to him an increased discharge might be accounted for in that way. If, in such a channel, it were possible thus to obtain without extra capital cost, an increase of 10 per cent. beyond what was anticipated, owing to the fact that silt was carried, it might mean that 10 per cent. of the gross revenue was added on to the net revenue. Even supposing the increase were only 5 or 7½ per cent., such a percentage of the gross revenue might be 15 or 20 per cent. of the net revenue, and that would suffice to turn an unprofitable enterprise into one paying handsomely. **Mr. Western.**

Mr. Lacey pointed out that in the Report of the Indus River Commission from 1906 to 1910, it was stated that a rising river had a higher velocity and discharge than a falling river for the same stage, and the reverse was the case for a water area, for there a rising river had smaller values than a falling one. He thought that rather supported Mr. Buckley's statement about the silt affecting the friction. **Mr. Lacey.**

He thanked the members for receiving his Paper so kindly. He was

Mr. Lacey. sorry that there had not been more discussion of it, but, being merely a résumé of matters he had observed in his professional experience, it did not lend itself much to criticism. The trouble in South India was the large-grain silt moving along the bed of the river. Looking at Dr. Deacon's experiments[1] it was possible to get an idea of how the silt was moved along the river-bed. A raised sill would not affect it very much, because when the shoal reached to the height of the raised sill the sand was carried into the channel, no matter what was done. The only thing was to widen the channel so as to get the required supply down. Many of the irrigation-systems in Southern India were reservoir-filling canals. The Palar and Pennair rivers had water about 80 days a year. The canals took off above those rivers and fed a chain of tanks, and the object was to get the flood-water down as soon as possible, and not to let anything go past. For that purpose a wide canal was necessary. The real solution would be some means of scooping a pool in front of the head sluices; how that could be done was a matter for the engineer. With regard to the erosion of one bank of rivers flowing north and south, for many years he had observed in India a straight canal which flowed north and south, and there steady erosion on the right margin and accretion on the left was noticeable. He could find no other reason for that except Ferrel's law, which might be recognized to apply to the movement of water also.

Mr. Rothery. Mr. ROTHERY, in reply, remarked that a maximum figure of 4·8 per cent. for the silt-content in suspension in the Colorado river was shown in the August, 1919, records, being equivalent to 48,000 parts per million; that was a rare occurrence, but a figure of 3 per cent. was frequently recorded. The mattresses in the weir were so placed to provide a spillway-length for the river-flow, which diminished to very small discharges during the late summer; consequently there was no overpour over a large portion of the weir. Downstream scour, or the washing out of brush bundles placed to close spaces, sometimes caused the settlement of a few mattresses during the season; in that event others were lowered into the breach or on top of those which had sunk. In places some rock from earlier rock-fill weirs existed, which resisted downstream scour and formed a silt foundation. No provision was made for the removal of the mattresses from the river for use in a second year; undoubtedly wedge-shaped prisms could be made and attachments provided for the cutting of the anchor-cable and for removal by means of the cableway. Much driftwood collected against the weir, and the brush units were so embedded in silt that the policy had been to use explosives to break up the units before the arrival of a high flood, which then carried all before it. As to the causes of the levee breaches, the sloughing away of the sides when the levees became saturated, due to side slopes being too steep for the plane of saturation, was a constant menace on those slopes that were not rock-revetted; and many sandbags were thrown into the semi-liquid places that began to show signs of flowing down. Breaches had happened from that cause, and from undercutting on the convex side of a river bend when flood conditions produced depths of 20 to 30 feet of water. The "Bell bund" had not been used on the Colorado delta, but the same result was obtained by dumping train-loads of rock around the bend while the waters were attacking the toe of the slope, the rock rolling down to the scoured river-depths. Overtopping had been the cause of levee failures on one or two occasions; a margin of safety of 3 feet above the previous year's high-water mark was generally sufficient to give 2 feet freeboard, allowing for the usual increase of 1 foot in flood height expected for the coming flood. Of course local conditions governed the height of the embankment along each levee. That would explain the omission of the high-water levels from *Figs. 7–9* of the Paper. The project for the building of a very high dam at Boulder cañon on the Colorado river had been examined by a Board of Engineers appointed to consider preliminary investigations, tentative designs, and estimates of cost. The Board had reported that they believed the dam to be physically practicable, and financially feasible, and had recommended that the United States Government should undertake the construction of a dam having a total height of 700 feet, of which 570 feet would be above low-water surface, in a narrow granite gorge, its base-length being only 200 feet, and its top length 1,130 feet. For its construction 2,800,000 cubic yards of concrete would be required, and the storage-capacity would be 26,500,000 acre-feet. A feature would be the hydro-electric development of 486,000 to 700,000 HP. The dam would undoubtedly be commenced before many years. He was glad to observe Mr. Buckley's reference to the fact that weeds did not grow below the water-surface in flowing silty water, due to the exclusion of light: that might have some bearing as a factor partly responsible for the higher velocities of flow required by silty water. Mr. Rothery.

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[1] Minutes of Proceedings Inst. C.E., vol. cxviii, p. 93.

Mr. Buckley. Mr. BUCKLEY, in reply, expressed his appreciation of Sir Murdoch Macdonald's remarks and of the value of his measurements of the discharge through the sluices of the Assuan dam. Some of Mr. Buckley's current-meter discharges of the Nile were

Mr. Buckley. checked[1] in 1918 and 1919 with the sluice experiments. He was indebted to Dr. H. E. Hurst for recently calling his attention to one instance of the phenomenon discussed in the Paper, with the river at a low stage under conditions identical with those which obtained in 1918, when the discharges were checked by the sluice experiments. Comparing the current-meter discharges from the 9th to the 24th July, 1921, with those of December of the same year, when the discharges in both periods were approximately equal, it was seen that in July the slope was 10 per cent. less and the hydraulic radius 6 per cent. less than in December, while the velocity was 5 per cent. higher; and whereas in December the bed was scoured, in July it was silted, showing that a higher velocity obtained with a silted bed than when scouring had taken place. Checking of the discharges by means of the sluices had not yet been achieved for the higher stages of the river dealt with in the Paper. This valuable check was only possible through the experiments which Sir Murdoch Macdonald had initiated and controlled. Measurements of discharges through sluices undoubtedly had practical applications where sluices were available, as long as the head was not too small, and as long as the conditions were not complicated by the velocity of approach. In the case of a river like the Nile the measurement of the water became an imperative necessity, especially when previously-uncultivated areas began to come under development. For the equitable distribution of the water, it was not only essential to know the volume discharged by the various tributaries, but also what proportion of their combined discharge was available along the course of the river—in other words, the losses or gains which occurred as it travelled along; and when it was remembered that the gain in the Nile due to return seepage from the banks between Khartoum and Sarras, a point not far south of Wady Halfa, had been estimated to be of the order of 25 per cent. of the Khartoum discharge when the river was low,[2] the importance of the discharge measurements became evident. As it was economically impracticable to construct barrages solely for the measurement of discharges, and as the cost of current-meter determinations was considerable, while the procedure was laborious and called for very careful supervision, it was important to develop, if possible, less costly methods, which would lend themselves to continuous determinations at any desired points from the sources of the river down to the sea.

He agreed fully with Mr. R. B. Buckley, that silt in suspension could exterminate weeds. Very little trouble was experienced in very deep drains in Egypt, and no weeds were found in the Nile, while it was only in some of the smaller canals, which were subject to strict closure during rotation periods, that weeds had been found to grow. In shallow drains weeds were a source of trouble. Moreover, very few canals in Egypt ever carried clear water. The silt records mentioned in Appendix C (p. 206) had shown that when the Nile water entering the canals was carrying only about 10 grams of silt per cubic metre of water, the silt-charge at stations near the tails of the canal-systems was sometimes as much as 500 grams per cubic metre of water. The increase, due to silt picked up by the water in transit, was more or less general in all the irrigation systems which were placed under observation. **Mr. Buckley.**

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[1] For the comparison see Minutes of Proceedings Inst. C.E., vol. ccxii, p. 228.

[2] Capt. H. G. Lyons, "The Rains of the Nile Basin." Cairo, 1905.

In thanking Dr. Unwin for his valuable observations, he felt that he was voicing the gratitude of all engineers who had to grapple with the measurement of water and the design of water-carrying channels. He appreciated fully the point of Dr. Unwin's remarks on the subject of the relative slopes of the water-surface and bed of a channel; a brief account of the evolution of the recorder might throw light on the problem. The Menufia Canal, the discharge of which was measured by the recorder, was one of the six main-feeder canals of the Nile Delta whose discharges were controlled from the Delta Barrage Office, of which he took charge some 11 years ago. He had found it a particularly unsatisfactory canal from the water-distribution point of view, because, unlike the others, it possessed a regulator at a distance of only 39 kilometres from the head, the manipulation of which seriously affected the canal slope and discharge, and also because in those days the discharge measured by current-meter or floats was only referred to the head gauge. He therefore erected a telephone-line along the canal for the simultaneous reading of two canal-gauges when the discharges were measured. Continuing these measurements from the end of March to the middle of October, 1913, he discovered that when the observations were made on an average once a week with slope-variations of the order of those shown in Fig. 1, Plate 3, the error between the observed discharges and those computed by the formula given on p. 200 never exceeded 6 per cent. As this result was obtained in spite of the objections raised by Dr. Unwin—which indeed had always been present in Mr. Buckley's mind—the result came as a surprise to him. The result could not, however, be disputed, and, having placed the facts before the Egyptian Government, he was authorized to proceed with the recorder for mechanically computing the discharges. Hence it would be seen that—at

Mr. Buckley any rate within the practical working limits of variations of slopes and levels in the canal, and in spite of certain theoretical objections—the recorder could be a practical proposition. In order to illustrate better the influence of varying water-levels when comparing the recorder discharges with those given by sluice or current-meter determinations, he had submitted with the Paper a reproduction of a weekly record diagram; this was filed at the Institution. It showed that an increasing discharge measured at 3 p.m. on the 28th September, 1921, differed from the recorder discharge at that hour by as much as 17 per cent. in defect; but the chart showed that, whereas the recorder began to register an increase at that hour, it did not indicate the correct value until 3 hours later: the instrument clearly required time to adjust itself, and a momentary comparison made when the levels were varying could be misleading.

Turning to the question of formulas in current use, while accepting Dr. Unwin's view that for certain stages of the Nile—as, for instance, when it was rising or falling rapidly—they might not be expected to apply, Mr. Buckley nevertheless felt justified in submitting that during the weeks when the Nile was fairly steady, say in September and October, 1921, referring to Fig. 11, Plate 3, the three formulas there discussed might have been expected to show considerably less departure from the true values, especially Kutter's formula, which was largely based on data collected from natural streams as well as from experimental channels. If Dr. Unwin's remarks referred more especially to the fact that the slopes used in the Paper were not local, that was, taken over a short distance of the river at the discharge-station, Mr. Buckley would remark that analogous results were produced by using the local slopes. For instance, the values of Kutter's coefficient of roughness were computed for the Baqlawis station for the flood period of 1919, using the local slopes, and were found to range from 0·024 to 0·064. It was therefore evident that Kutter's formula could not have been applied with satisfactory results, even with the use of the local slopes, without varying the coefficient of roughness. This was confirmed by the results obtained on the Indus, referred to later in replying to Mr. Lacey.

He much regretted that apparently some of his remarks might lend themselves to an interpretation somewhat disparaging to existing formulas, as nothing had ever been farther from his intentions; but it might be of use to point out to engineers, who from time to time had to use those formulas for lack of others for determining flood-discharges of rivers—as, for instance, had been done in the United States—that where much silt was present in suspension, unless care had been taken to determine the coefficient of roughness

Mr. Buckley.

for several stages of the river with varying silt loads, the computed discharges might be in error by as much as 50 per cent.

Dr. Unwin appeared to doubt, owing to the known turbulence of water, the damping action with respect to eddies of heavily silt-laden water at the bottom of a stream; but repeated soundings had shown the existence of liquid mud or slurry of very heavy fluid consistency at the bottom of the Nile in flood. The "Encyclopedia Britannica"[1] contained the following passage, which appeared to Mr. Buckley to have an important bearing on the subject. It referred to the causes which contributed to the resistance to flowing water:—

> "Volumes of fluid are detached continually from the boundaries, and, revolving, form eddies traversing the fluid in all directions, and sliding with finite relative velocities against those surrounding them. *These slidings develop resistances incomparably greater than the viscous resistances* due to the movements varying continuously from point to point. The movements which produce the phenomenon commonly ascribed to fluid friction must be regarded as rapidly or even suddenly varying from one point to another. The internal resistances to the motion of the fluid do not depend merely on the general velocities of translation at different points of the fluid (or what Boussinesq terms the mean local velocities), but rather on the intensity at each point of the eddying agitation."

It seemed evident that slurry, or liquid mud as referred to above, must of necessity be less liable to eddying agitation than clear water. Dr. Unwin thought that inconsistency had sometimes been shown in the Paper with regard to the above theory, owing—if Mr. Buckley understood him correctly—to the fact that the coefficient in the Beleida formula depended entirely on the number of grams per cubic metre of water; but Mr. Buckley had hoped that the wording of the second sentence on p. 197 would show that the silt in suspension had been taken as a measure of the consistency of the slurry for lack of any better means of gauging it. The difficulty of doing this would probably be clear, but it seemed reasonable to infer that in a river flowing under natural conditions, like the Nile, there might be some relation between the two, although, as stated on p. 197, the relation might not be the same for all rivers. The case of an artificial channel, to which the reference on p. 187 to which Dr. Unwin had alluded applied, such as an irrigation-canal designed to run at a definite slope and frequently taking only that part of the silt-laden mixture nearer the surface of the parent stream, was somewhat different. In many such cases the mixture found itself in a channel of good silt-carrying

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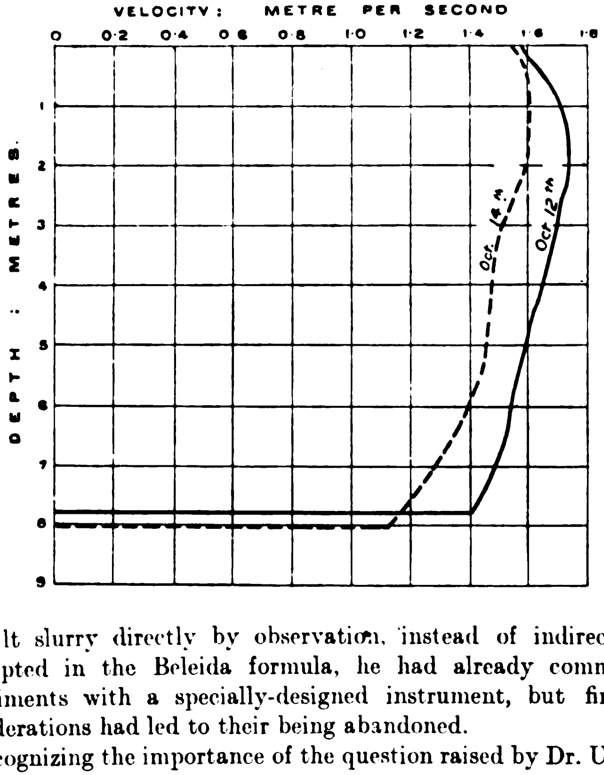
[1] 11th ed. 1910 Vol. xiv, p. 35. Art. HYDRAULICS (*Measurement of Viscosity.*)

Mr. Buckley. proportions where no silt-deposits need occur, especially in Egypt, where the silt-load was comparatively small, and where the presence of silt in suspension would probably have no influence in diminishing the rugosity of the channel. In India, on the other hand, where the canal-slopes were considerably greater and greater silt-loads were carried, it appeared from Indian records that the phenomenon had been observed also in canals. The last two sentences at the foot of p. 200, with the text at the top of p. 201, also dealt with this question.

With regard to the possibility of determining the consistency of the silt slurry directly by observation, instead of indirectly as attempted in the Beleida formula, he had already commenced experiments with a specially-designed instrument, but financial considerations had led to their being abandoned.

Fig. 1.

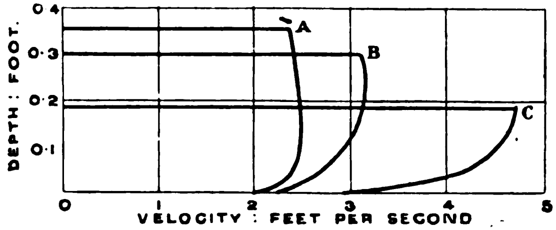


Recognizing the importance of the question raised by Dr. Unwin's request for vertical velocity curves, he had included with the original manuscript of the Paper a complete set of such curves, which was filed at the Institution. However, as explained on p. 189, the velocity at the periphery of the channel when it was silted was greater, when compared with the maximum velocity, than when

Mr. Buckley.

the section had scoured. The curves of vertical velocities at all the buoys, with the notable exception of buoy No. 7, referred to in the Paper, showed this effect more or less. The curves for buoy No. 5 were reproduced in *Fig. 1* (p. 238), with their water-levels plotted on the same abscissa. This diagram should be compared with *Fig. 2*. In *Fig. 1* the curves A and B diverged at the bed of the channel; in *Fig. 2* they diverged at the water surface. There appeared to be only one explanation, namely, that while the curves in *Fig. 2* owed their profile to varying velocity induced by change of slope, those in *Fig. 1* owed their profile to reduced eddying agitation due to the silt. While this was a graphic demonstration of the phenomenon for one buoy only, the numerical demonstration referred to above and on p. 189 showed that the condition of net increased velocity at the bed was general for the whole section of the river. Mr. Beresford appeared to think that the increase in velocity would lead to the silt being picked up. But, as explained later (p. 242), the carrying-capacity of a channel seemed to depend on the rate of change of velocity along a vertical as well as on the absolute velocity; hence, if by reducing the roughness of the bottom the films of water nearest to it had their velocities increased in a greater degree than those at higher levels, the rate of change of velocity in verticals became reduced and with it the silt-carrying capacity of the stream, which would eventually require a considerably greater mean velocity to pick up the silt again.

Fig. 2.



Slope for A *0*
" " B *0·26%*
" " C *0·56%*.

The question of variable flow had been gone into some years ago in connection with its probable influence in computations of the Nile discharges from water-slopes, before the silt effect had begun to materialize as a possible governing factor. The conclusion then come to, and confirmed by a more recent investigation, was that the term *dv/dt* had no significant value for the Nile.

Mr. Buckley. He found it a little difficult to follow the "smooth surface" hypothesis as enunciated by Professor Dixon. What was meant by a "smooth surface"? Did it mean a smooth plane surface devoid of undulations? That would be the only effective eddy-reducing form of surface, as any undulating surface, however smooth, would still cause retardation of flow on account of the eddies thrown off by the undulations, projections, etc., of the bed. On this aspect of the problem he would point out that in his experience of the Nile no such plane surface could be said to exist. He hesitated to accept Professor Dixon's statement that the Paper proved that silt was deposited in the inequalities rather than elsewhere. The Paper showed that deposits measurable with the flat-bottomed sounding-lead occurred; but it must be remembered that the point at which the lead was arrested was the surface of the coarser sand, which was generally itself travelling at a low speed, probably in the form of sand-waves, as for instance found by the late Dr. G. F. Deacon, M. Inst. C.E., in his experiments,[1] the sand-waves themselves travelling forward over the original inequalities of the bottom. Thus, in his conception of the conditions, there existed no definite surface created by silt deposits as required to satisfy Professor Dixon's hypothesis as he understood it. On the other hand, the existence of a travelling stratum of slurry had been frequently proved, and the simple experiment of casting a stone into it, as could be done when an irrigation-canal was first being dried off, showed that waves were not propagated along its surface as they were in clear water. Professor Dixon's hypothesis, therefore, did not appeal to him, as it appeared to envisage certain conditions which he thought did not actually obtain. However, the important point was Professor Dixon's apparent agreement that the presence of silt did modify the conditions of flow.

With regard to the determination of the hydraulic radius, he would refer to Sir Murdoch Macdonald's Paper.[2] Some of the current-meter discharges which were embodied in Mr. Buckley's Paper were there compared with the sluice-discharges based on direct measurement into a tank; the mean of the differences was of the order of 1 per cent., and it seemed improbable that if any of the factors were greatly in error such consistent agreement could exist between the determinations. Professor Dixon's determinations by means of soundings taken through holes in the ice seemed hardly comparable with the observations taken on the Nile, where the sounding-lead was kept vertically below the boat by means of the stay wire. Secondly, the curves of area and hydraulic radius plotted against the Nile gauge where the slope was not subject to abrupt variations showed marked regularity in the diagrams; a regularity which one would expect to be absent if the method of determining the quantities were liable to considerable error. In connection with this matter it should be pointed out that just as varying the area of sluice-opening could affect the sectional area of the river, for reasons explained in the Paper, so could the mere presence of the dam have the same effect at the higher levels of the river, when the discharge was fluctuating, owing to the obstruction which the reduced discharge section of the open sluice presented to the flow of the river, creating a natural head of 2 metres when the sluices were all open to the full extent. Hence, the variations in the sectional area of the discharge section could in part be attributed to this cause, especially when the river was heavily charged with silt at the peak of the flood, and the existence of these irregularities in the curves of *R* and sectional area at the highest stages of the river could thus be largely accounted for. As to possible errors, due to the positions of the soundings being fixed by box sextant observations, reference to Fig. 4, Plate 3—which was plotted to a much-distorted scale—would show that differences of the order of 10 metres on either side of the position as located would have made no appreciable difference in the result; in this connection the general close agreement between the positions of the soundings on the 12th and the 14th October, as shown plotted on the section, might also be remarked. Moreover, by sighting on fixed beacons on the banks, the discharge boat was very accurately aligned on the discharge section, the sextant observations being required only for determining the distance from the base. Mr. Buckley.

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[1] Minutes of Proceedings Inst. C.E., vol. cxviii, p. 93.

[2] *Ibid.*, vol. ccxii, p. 265.

Professor Dixon's references to Mr. G. K. Gilbert's experiments[1] were of much interest. It seemed essential, however, to make quite certain that, in comparing the results of experiments, the conditions were always the same. The experiments in question were carried out in three wooden troughs, the longest of which was 150 feet in length and 1·9 foot wide, while in some, a fourth trough, made of iron, was used. It was explained that in carrying out the experiments with added solids in suspension—the solids being added at a given rate—they at first formed a deposit along the bed of the trough until a stable section was attained, and the measurements of the various elements were then made and recorded. Hence, when the velocities

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[1] "The Transportation of Débris by Running Water." U.S. Geological Survey: Professional Paper No. 86. Washington, 1914.

Mr. Buckley. of "loaded" and "unloaded" experiments were compared, as in Professor Dixon's quotations, it appeared that the conditions of the bed were not the same, for in the case of the unloaded experiments the water was flowing against the surface presented by the bare boards of the trough, whereas in the case of the loaded experiments, the water was flowing over a sandy bottom having a sand-wave or "dune-like" formation, as sometimes described in Mr. Gilbert's publication. Now, it was very interesting to find that Mr. Gilbert recorded that "the reduction of mean velocity by traction is greater for small loads than for large." The conditions of the trough surface were similar in the loaded experiments, but Mr. Gilbert found that the retarding effect was less with the greater loads; this conclusion seemed to be identical with that expressed in Mr. Buckley's Paper.

But the question arose whether Mr. Gilbert's account was truly descriptive of the phenomena, and whether the load had a retarding effect, or whether it would be more correct to say that under different conditions, to which Mr. Gilbert also alluded, the stream could carry a greater load. Thus Mr. Gilbert stated that ratio of depth to width of a stream, and diversity of velocity in a stream were factors which affected its silt-carrying capacity. The former corresponded to Kennedy's theory, the latter had been stated by Dupuit, Flamant and others, namely, that the stream with the best carrying-capacity was that in which the tangent to the vertical velocity-curve made the most acute angle with the bed; for in that case the rate of change of velocity from one horizontal film of water to another was greater than in a stream having a more vertical curve. Taking any recorded hydraulic experiments—as, for instance, Bazin's, where he showed a deep triangular section and a very shallow wide section, the discharge and slope being practically identical—the direction of the curve of velocities in a vertical for the deep section was practically perpendicular for three-quarters of the depth measured downwards from the surface. The curve was very similar to curve A of *Fig. 2* (p. 239); in that portion of the section the carrying-capacity would therefore be low, as the rate of change of velocities in a vertical was insignificant. But, in the case of the wide and shallow section, the velocities diminished rapidly from the surface of the water down to the bed; with this shallow section the carrying-capacity was good. In both experiments the maximum velocity was the same.

The velocity-curves of Mr. Gilbert's experiments with clear water exhibited the same characteristics. *Fig. 2* was a reproduction[1] of

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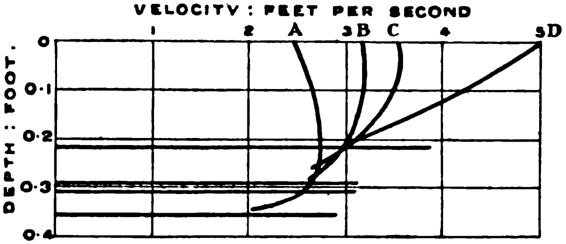
[1] The Figures from Mr. Gilbert's Paper are reproduced by permission of the U.S. Geological Survey.—A. B. B.

Mr. Buckley.

the curves in Mr. Gilbert's Paper. The discharge was the same in every case, but the slope of the trough was set to the values shown. It was evident, therefore, that for a given discharge the channel corresponding to curve C, with a large value for the ratio of bed-width to depth, had the vertical-velocity curve whose tangent made the smallest angle with the bed. This was instructive, on account of all three experiments having been carried out in the same channel, thus eliminating the possibility of the result being influenced by varying degrees of roughness of the bed. Therefore, in accordance with the hypothesis under review, the A curve showed conditions of bad silt-carrying power, while of the three conditions that shown by curve C would be the best for the given discharge. His reply to Mr. Elsden (p. 245) also dealt with this point.

Considering next the velocity-curves in Mr. Gilbert's experiments

Fig. 3.



Curve	A	B	C	D
Load : Grams/Second.	0	38	53	194
Depth : Feet	0·353	·313	·303	·226
Slope of Bed : per cent.	0	0·38	0·56	1·14

under loaded conditions, *Fig. 3* showed Mr. Gilbert's curves for different loads. As already mentioned, the method followed in carrying out the experiments was to add the load at a constant rate, allow the channel to silt up until stability was reached, and then observe the various elements. *Fig. 3* showed clearly that for every increase in load the channel adjusted itself to a shallower section, so as to give a greater value to the ratio of bed-width to depth. This confirmed Kennedy's theory. Moreover, the curves showed that the heaviest load was carried when the tangent of the velocity-curve made the most acute angle with the bottom of the channel. Mr. Gilbert seemed to attribute the form of the velocity-curve chiefly to the load. Mr. Buckley, on the other hand, held the opinion, in agreement with the authorities already cited, that the form of the velocity-curve was chiefly influenced by the ratio of bed

Mr Buckley. width to depth: at the same time, he had come to the conclusion, as explained in his Paper, that the form of the curve might be modified within limits by increasing the density of the fluid nearest the bed where the latter had a corrugated or dune-like formation. Therefore, in Mr. Gilbert's experiments he found corroboration of the theory discussed in his Paper, as well as of Kennedy's theory. He would emphasize how much, in his opinion, contemporary literature had been enriched by Mr. Gilbert's Paper, from which engineers could obviously obtain much enlightenment.

Sir Thomas Ward's opening remarks were much appreciated by Mr. Buckley and no doubt would be by other workers who, while also conscious of their limitations, made an effort to put their experiences and conclusions on record. Having been responsible for the design and partial execution of projects involving the remodelling and construction of waterways commanding an aggregate of some 4,000,000 acres, Mr. Buckley had been brought into intimate contact with the difficulties of deciding what types to adopt for non-silting canals, and in choosing coefficients to be applied in the formulas in current use for determining their discharges. He had had opportunities for personally inspecting the canals of the Punjab, both when running full and when dry. He had been particularly struck by the fact that there appeared to be no evident reason for the large differences in Kutter's coefficient of roughness applicable to canals in Egypt and in India. The nature of the soil as regarded rugosity and the regularity of the sections, seemed much the same, and, to take a specific case, it was not obvious why the Rakh branch of the Lower Chenab canal should have a Kutter coefficient of roughness of 0·016 to 0·018, while the Menufia canal in Egypt had a coefficient of 0·0275. Differences of this order, to the practical engineer, appeared in the form of capital expenditure, and were therefore thoroughly brought home. Mr. R. G. Kennedy's pioneer work had achieved great economies in India, and had resulted in producing canals which kept themselves free from accumulations of silt. That did not necessarily mean that the canals did not silt temporarily at certain seasons. Reference to Indian authorities on the subject showed that the coefficient of roughness was greatly diminished when canals were silted, but no definite explanation seemed to have been put forward to explain this result, although there did appear to have been an underlying idea that silt deposits smoothed the bed. It was well known, however, that heavy silt travelled along the bed of a channel and did so in the form of transverse corrugations or sand-waves, and there could therefore be little doubt that the roughness-reducing influence of the silt-laden water was not brought about by the creation of a smooth plane surface. **Mr. Buckley.**

Mr. Elsden's reference to a temporary reduction of resistance to flow during a period of high silt-content confirmed Mr. Buckley's contention that, if irrigation canals were equipped with proper recording-devices, much valuable evidence could be accumulated, which would lead to a better understanding of the general conditions governing the flow of water. Phenomena of the kind referred to by Mr. Elsden usually occurred with abnormal conditions of régime, while, owing to the impossibility of predicting their occurrence, it was generally impracticable to make arrangements for carrying out the necessary observations. Mr. Buckley agreed that in order to avoid the silting of canals it was necessary to design headworks so that grains greater than a specified size should not be admitted, and that this could be attained only by drawing the water of the parent stream, by suitably-constructed headworks, from the upper levels down to a point where the silt-carrying conditions were such that the larger particles were not in suspension. When once this fact was established, no engineer need have any doubt as to the possibility of keeping a canal free from silt as long as the flow was maintained. With his Paper entitled "Irrigation Canal Headworks," Mr. Elsden had made a valuable contribution to the literature on the subject of silt-carrying channels. He referred therein to Mr. F. W. Schonemann's Paper entitled "The Design of Head Regulators to Irrigation Tributaries" read at the Punjab Engineering Congress in 1916; and there could be no doubt that in bringing into prominence Dupuit's hypothesis, namely, that the silt-supporting power of a stream was a function of the rate of change of velocity in a vertical, Mr. Schonemann has rendered a very useful service.

He agreed with Mr. Beresford that it was not strictly necessary to read the gauges to millimetres: on p. 203 he explained why the water-levels had been so read. He felt, with regret, compelled to dissent from the statement that it had been thought in India that silt-laden water would have a lower velocity as compared with clear water, as standard works by authorities with Indian experience contained references to the contrary. For example:—

P. A M. Parker. "The Control of Water":—"As a general rule it may be stated that water which carries fine silt can be expected to show a somewhat lower value of '*n*' than clear water under similar conditions."

Mr. Buckley. E. S. Bellasis. "Hydraulics":—". . . over a hundred discharges observed near the head of a large canal in India . . . show the average value of 'n' to be 0·025 when there is no silt, but 0·013 when the depth of silt is 0·5 foot upwards . . ." (See R. B. Buckley's "Irrigation Pocket Book.")

While agreeing with Mr. Beresford that colloidal particles might be held in suspension irrespective of the velocity, he would point out that the statistics contained in his Paper showed that in the Nile there existed a stratum of heavier fluid or slurry near the bottom, composed of particles of varying size. As to the figures on p. 189, he had already explained, in replying to Dr. Unwin, why an increased velocity near the bed might be compatible with a blanketing layer of slurry to which it might owe its origin. He much regretted that the wording of the Paper was not as clear as it should have been, and seemed to have led Mr. Beresford to attributing the washing away of the silt to an increased velocity, whereas Mr. Buckley had endeavoured to show in the last paragraph of p. 187 that the blanket of slurry, as well as some of the heavier deposit which could be detected by means of a sounding-lead, had been washed away in the period 12th–14th October, when the mean velocity of the river was reduced; and he had further attributed this phenomenon (p. 187 with a further explanation on p. 191) to the abstraction of silt from the Nile water by the silt having been deposited in the Assuan reservoir, owing to the manipulation of the sluices for maintaining a given level in the reservoir. In other words, the water issuing from the sluices, having been deprived of the silt which the existing conditions of velocity, as well as ratio of width to depth of the river, enabled it to carry, endeavoured to restore the amount of silt in suspension by picking it up from the bed of the river downstream of the dam, thereby restoring the section to the original unsilted profile as shown by Fig. 3, Plate 3. The net result of the operation was to reduce the mean velocity of the river; and in spite of this reduction it was still able to carry the silt which it had picked up: this suggested that the total charge as picked up was less than the charge before the régime of the river was disturbed by the partial closing of the sluices of the dam. He would put the case thus:—The Nile flood, which in its initial stages was entirely due to the Blue Nile, as it advanced, laid a blanket of silt on the bed of the river from the Abyssinian mountains to the sea. The particles of silt composing this blanket need not necessarily have ever been in suspension nearer the surface of the river; the slurry might have existed as such from the original source of the silt, and this was probably the case. As the Blue Nile flood subsided, the

Mr. Buckley.

proportion of the comparatively siltless White Nile contribution, which at the crest of a high Blue Nile flood was insignificant, became greater and greater. The Nile, flowing at any given level with a given slope, had a definite avidity for silt, and when the contribution of the comparatively siltless water from the White Nile began to have a preponderating influence, the silt blanket was picked up to satisfy the river's appetite for silt. If now the silt conditions were still further modified by causing silt to be deposited in the reservoir, the river, in order to maintain its normal silt-load, would at once pick up the silt it required until no more was available; and this appeared to be what occurred in the period referred to on p. 189. In other words, the conditions which obtained on the 14th October were governed, not so much by the velocity of the water, as by the reduction of the charge of silt in the water downstream of the dam, due to retention of silt in the reservoir.

Referring to Mr. Beresford's remark on the load-carrying capacity of a velocity of 1·7 metre per second, as Mr. Buckley believed that the carrying power of a stream with a given width was a function of the slope, depth, irregularity of the bed, and degree of fluidity of the layers adjacent to the bottom, he hesitated to agree that it could be specified by the magnitude of the velocity alone.

He had already dealt with the question of the accuracy of the soundings, and he would mention here the independent current-meter determinations of the same discharge referred to on p. 203, which yielded results agreeing within 2 per cent., as well as the comparisons between the current-meter and sluice-discharges referred to in his reply to Professor Dixon. It seemed improbable that any of the observed quantities in those determinations could be in error by the figure of 10 per cent. given by Mr. Beresford. These checks could not be applied to the higher stages of the river, but occasions were not lacking for observing the silt phenomenon during the lower stages, as mentioned in reply to Sir Murdoch Macdonald. It appeared that the Nile, owing to its comparatively low silt-load, was not one of the best rivers for this kind of research. It was hoped that the Paper would be followed by further research, but meanwhile Mr. Lacey had kindly drawn attention to a similar instance commented upon in the Indus River Commission's Report. The Indus carried approximately twice as much silt as the Nile, and the silt influence appeared to be of the order of 40 per cent.; and possibly the limits were considerably greater.

Mr. Western's remarks were particularly welcome because, for some considerable time, the hydraulics of open channels appeared to have been accepted as a more or less hopelessly empirical science, Mr. Buckley. almost unworthy of that name, with little prospect of ever becoming anything better. No doubt many others were steadily working at the problem, and it was most interesting to know their views on possible lines of new investigation. Owing to the great width of the Nile and its depth, it had not been found possible with available means to devise any method of comparing the relative levels of the surface of the river, except at the banks. Slight differences in level between one side of the river and the other had been observed. The problem was one which had been recognized as calling for careful investigation, but it had not been found possible to tackle it. Something had been done to explore the variation in velocities at fixed points and so on, but no conclusions sufficiently definite to warrant publication had been arrived at. As mentioned in the text, the Paper was simply a statement of progress of work which had had to be brought to an abrupt termination, published for general information. The question of the probable increase in velocity due to increased specific gravity had been gone into, but in view of the fact that in the Nile the maximum increase probably amounted to about 1·5 per thousand, it did not seem likely that it could have a preponderating influence.

He thanked Mr. Lacey for his reference to the Indus River Commission Report, which was clearly of importance. The reference was to the Kotri discharge-station, which was described as favourable for the required observations. The report contained complete records of the hydraulic elements as collected for Mr. Buckley's Paper, and was in consequence particularly interesting. The passage to which Mr. Lacey referred concerned the discharges of the Indus on the under-mentioned dates, and pointed out as remarkable that when the river was falling the mean velocity for similar slopes was less than for the rising river, although, with the falling river, the cross section was greater. As, with the reference to this example, all the hydraulic data which entered into the problem were not tabulated, he has extracted them from other sections of the report and reproduced them below, after converting them into their metric equivalents for comparison with the rest of his Paper.

Date.	Height of Gauge.	Slope.	Mean Velocity.	Hydraulic Radius.	Area of Section.	Discharge.	Remarks.
1909.	Metres.		Metres per Sec.	Metres.	Sq. Metres.	Cubic Meters per Sec.	
16 June	4·66	0·000064	2·128	3·97	2,949	6,275	River rising.
28 Sept.	4·69	0·000060	1·192	4·95	4,288	5,114	River falling.

Increase in $R = 25$ per cent.

Mr. Buckley.

Reducing the velocity on the 16th June to the equivalent which would obtain with a slope of 0·00006 as on the 28th September, its value became 2·060 metres per second; thus the velocity on the 28th September represented a reduction of 42 per cent. on the former velocity, in spite of identical slope, and an increase of 25 per cent in *R*. The report showed definitely that the section had scoured. Other conditions being the same, it was difficult to see to what such a heavy reduction could be attributed, other than to increased resistance to flow due to eddying agitation arising from the scoured bed of the river. A curve of values of Kutter's *n* was also given, whence it appeared that the limiting values were 0·009 when the river was heavily charged with silt, and 0·039 when the reverse was the case.

In conclusion he tendered his best thanks to the Council of the Institution for the acceptance of his Paper, and to those who had taken part in the discussion. He was glad of this opportunity of expressing his gratitude to Messrs. J. S. Wilson and S. Lelyavsky, to the former for taking great trouble in displaying lantern-slides at the meetings, and generally making an all too imperfectly drafted document a little less obscure and unattractive, and especially for his interest in the research, accompanied by valuable advice during the 11 years preceding the publication of the Paper. To Mr. Lelyavsky his sincere thanks were due for help in dealing both with mathematical problems and in suggesting experimental methods, while his extensive knowledge of British and foreign literature on hydraulics, and of languages, had permitted of a more thorough study of sources than would otherwise have been possible.

Correspondence.

Mr. Alford.

Mr. J. S. Alford was of opinion that the statement in Mr. Lacey's Paper which referred to the influence of the rotation of the earth on the erosion of river-banks was not quite adequate. That influence was not more potent in rivers flowing in a north and south direction than in others. In the northern hemisphere rivers flowing north were continually arriving at a latitude of lower rotational velocity. As the earth was rotating eastward, the excess velocity was disposed of by pressure against the east or right bank. Rivers flowing south passed to zones of higher velocity and had the additional velocity imparted to them by pressure from the west or right bank. The water in rivers flowing east travelled faster than the banks and had a tendency to press towards the equatorial or right bank. In rivers

Mr Alford. flowing west the water, moving more slowly than the banks, had a tendency to move towards the pole, and pressure occurred between the polar or right bank and the water. In the southern hemisphere similar causes produced pressure on the left bank. Rivers which were not flowing towards one of the cardinal points of the compass partook of both influences. Professor W. Ferrel's writings[1] dealt with the subject, principally in regard to the movements of atmospheric air, but his conclusions applied also to rivers, and were generalized so as to apply to all rivers, whatever the direction. The pressure per square foot of river-bank was proportional to the breadth of the river, to the velocity of the water, and to the sine of the angle of latitude. The greatest effect, dealing only with that aspect of the force, was to be expected in large rivers in high latitudes when in flood. Indian rivers, on account of their low latitudes, were not likely to be conspicuous examples. The case of the Colorado river breaking into the Salton sea was pointed out by Mr Lacey, but Mr. Rothery, who referred in detail to the break-through, did not mention the earth's rotation as a contributory cause. To find the effect at its greatest it would seem necessary to turn to the great Siberian rivers. The Yenesei reached the sea about latitude 73 degrees. Dr. F. Nansen, in his book on Siberia,[2] describing his journey up the Yenesei, the deep-water channel of which was on the right or eastern side, referred to the remarkable difference between the right and left banks of the river, and stated that there was no getting away from the conclusion that such was due to the earth's rotation. Dr. Nansen regarded the geological structure as being an additional factor. The question had been asked, Why, if the theory was true, was not a larger area of alluvium found on the left than on the right bank of rivers of the northern hemisphere? Some rivers in relatively low latitudes did show that characteristic. The reason why it was lacking on others might be the following:—Forces which tended to cause excess pressure against one bank also tended to direct towards that bank all silt being carried down the river, the specific gravity of which was greater than 1·0. The latter action made for accretion, while the former was erosive. Whether the resultant sum was plus or minus would appear to depend on the circumstances of each case, and particularly on the quantity and nature of the silt brought down by floods. The deflection of the silt was an influence superimposed on all centrifugal forces set up by curvatures of the river course. Captain H. G. Lyons, in his book on the Nile,[1] recognized the existence of centrifugal force as a factor, as was evident from the following extract:—"Throughout almost the whole of its course the Nile in Egypt flows on the eastern side of its valley and while the earth's rotation has played its part in deflecting the stream to the right, the action of the wind has probably been still greater." **Mr. Alford.**

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[1] "Recent Advances in Meteorology." Annual Report of the Chief Signal Officer of the U.S. Army, 1885, Part 2, and "Popular Treatise on the Winds." London, 1889.

[2] "Through Siberia," pp. 72, 140. London, 1914.

Mr. Buckley. Mr. A. B. BUCKLEY observed that the ground covered by Mr. Rothery's Paper was so extensive that the subject of each section would provide scope for years of work for any individual. The Paper showed clearly that the rate of accretion over certain areas might be very considerable: from that arose the conclusion that the Colorado river was still in the process of forming its delta, and in that respect appeared to be many years behind similar rivers in other parts of the world. Comparison of the maps of the United States and Mesopotamia showed that the distance from the foot of the Chocolate mountains in Arizona to the southern end of the gulf of California was comparable with the distance from the foot of the hills of Kurdistan to the end of the Persian gulf; but, whereas the Tigris and Euphrates had filled up a distance equivalent to about half the length of the gulf of California, the Colorado river appeared to have only begun its task. Did the explanation lie in the fact that the Colorado river, whose silt-charge was comparable with that of the Tigris, had already levelled off other large areas to the north of Yuma? The subject was of great interest, and, no doubt, an explanation would be forthcoming one day, with the help of Papers like Mr. Rothery's. The Nile provided several similar examples. It was generally accepted that the Nile valley, from Assuan to the sea, was the product of the river, and that in remote ages it had constituted a gulf more or less similar to the gulf of California as it existed to-day, only very much narrower. But the Nile valley from Assuan to the sea was not the only product of the river. The Blue Nile had created its delta from Roseires to Khartoum, and there was evidence that in carrying out its task it had gradually pushed the White Nile farther west against the high desert, which had now arrested that lateral displacement. The Atbara, another tributary of the Nile, had done the same. It had made its channel, and its water now flowed into the Main Nile at Atbara town. On the other hand, the Gash, another would-be tributary of the Nile, was a river which, like the Colorado, was for some unexplained reason still at the beginning of its task; it carried

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[1] "The Physiography of the River Nile and its Basin." Cairo, 1906.

Mr. Buckley. large quantities of silt, its slopes were very considerable, judging by the high velocities of its flow, and at present, although making for the Nile valley, it lost itself in the desert and must continue to do so until, after having levelled off a sufficiently extensive area, it should have formed for itself a channel, as had happened with the Blue Nile and the Atbara, which had sections of such proportions that they could carry their silt-charges. To deal with the canalization of rivers like the Nile in Egypt or the Blue Nile, both of which might be classified as stable, was a comparatively simple problem, while it was obvious from Mr. Rothery's Paper that to deal with rivers of the other kind was a far from simple matter. As to flood-protection difficulties, he considered, as a result of personal experience, that the Colorado river, in the districts dealt with in Mr. Rothery's Paper, was in many ways similar to the Tigris and Euphrates. An explanation was frequently sought for the numerous derelict canals which abounded in Mesopotamia; in his opinion the cause was the same as that which brought about the bankruptcy of the irrigation corporation mentioned by Mr. Rothery. There could be no question as to the wisdom of building no permanent regulating-work across the river at Yuma. An instance of recession of a river could be seen at the present day on the Euphrates upstream of Samawa, due to the same cause as that which created the same effect between the Colorado river and Salton lake. The only truly safe method of flood protection for rivers like the Colorado or the Tigris and Euphrates, and possibly others, could be found in flood-storage reservoirs, unless those also would be liable to obliteration by silt-deposits. In that connection silt-determinations for the Colorado river at Boulder cañon would be of interest. The difficulties presented by the flood-protection problem were evidently great, but they were being dealt with efficiently. He would be interested to see data concerning the river's capacity for silt-transportation collected on the same lines as those followed in Egypt and alluded to in his own Paper. The ultimate height of the protection-levees must be governed by the probable ultimate level of the river, but it seemed difficult to arrive at any approximate value without much more information than was available at present. It did not appear necessarily to follow that the flood-level must increase to any prodigious extent above the present maximum recorded, as long as sufficient care was taken to train the river to its proper section, which must depend on the silt-load and the minimum slope required to carry it. The distance from Yuma to the open waters of the gulf was approximately 100 miles—a very short distance in comparison with those dealt with in other river projects. It would

Mr. Buckley. appear to be advisable to embank the river on both sides, to avoid the relative slackening of flow which must occur when the river began to spread over the plain. It was usually under such conditions that the silt-deposits were formed which caused obstruction by shoaling with excessive flood-levels. The design of the Rockwood gate headworks seemed to be admirably adapted for the exclusion of heavy silt from the main canal. It would be of interest if Mr. Rothery would say whether care was always taken to admit water through every opening at the highest possible level, so as to exclude the heavier sand loads. The brush-mattress weir of original design appeared to be most adapted to the locality; it was a type which should prove very useful under the same conditions in other countries. The reference to the water-logging of 120,000 acres was of interest; unfortunately it was a feature which was not uncommon. Subsoil-water statistics were usually scarce, and if Mr. Rothery could publish data of water-table levels from year to year, and state the locality of the damaged area, the information would prove instructive. In some cases, as for instance in the Nile valley, the rise of subsoil water might be due as much to the high level maintained, on a barrage across the river for feeding the canals, as to the irrigation-canals to which the seepage was usually attributed. In such cases drains were of little assistance unless very closely spaced. He complimented Mr. Rothery on his concise and instructive Paper.

Mr. Butcher. Mr. A. D. BUTCHER observed that the questions raised by Mr. Buckley's Paper were undoubtedly of great importance to irrigation-engineers. Mr. Buckley's contention appeared to be that, other factors being equal, the velocity of flow in a given section was increased by the presence of silt in the water. Unfortunately, Mr. Buckley, in one portion of his Paper, appeared to consider that the silt in suspension affected the velocity, while in another place he contended that the effect was obtained only when the silt was in process of being deposited or lifted. The Beleida formula was certainly based on the former supposition, while most of the general argument was concerned with the latter. The Paper dealt with the relation between velocity, hydraulic mean radius, and hydraulic slope, and with the possible modification of that relation due to silt. The hydraulic mean radius in the Khannaq and Beleida experiments had been determined at one section only, while the hydraulic slope was that over a long reach—Assuan to Khannaq, 26 kilometres, and El-Leisi to Beleida, 14 kilometres. The hydraulic slope over a long reach could not fairly be applied to a single section at one end unless that section bore a fixed relation to the mean section over the reach. The rapid changes in individual sections which occurred in

Mr. Butcher. the Nile (see Fig. 4, Plate 3, where the section scoured 1·5 metre in 2 days) made it impossible that a single section could represent adequately a long reach; and the method employed of applying the mean hydraulic slope to a single section, therefore, would lead inevitably to erroneous results. In this connection it might be noted that Mr. Buckley attempted the determination of local water-slopes at Khannaq, and that those slopes differed widely from the mean slopes finally adopted. For example, on the 26th March, 1920, at Khannaq the hydraulic slopes over two successive kilometres were reported as:—

Up-stream kilometre	0·5 centimetre per kilometre.
Down-stream „	4·27 centimetres per kilometre.
Mean slope, Assuan to Khannaq . .	5·1 „ „

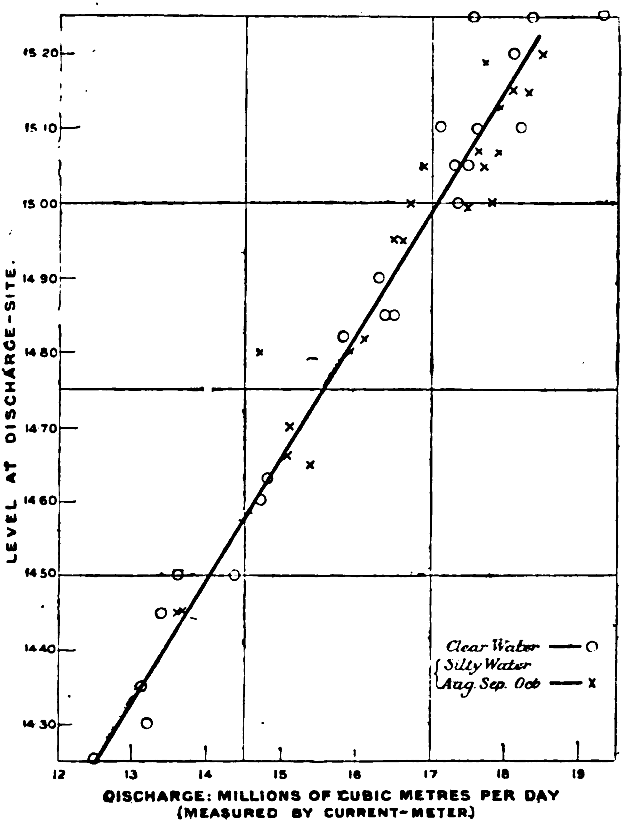
Many other similar instances might be cited. The very large difference between the local slopes needed explanation, but it was obvious that, had either of the values been used in computation, the results obtained would have differed widely from those resulting from the use of the mean slope from Assuan to Khannaq. On those grounds alone the conclusions regarding the action of the silt in modifying the relation between velocity and hydraulic slope should only be accepted with the greatest reserve. The Beleida formula accurately represented the observed data: that was in itself, however, no proof of the general contention that silt in suspension actually affected velocity, as a formula could be arrived at which took no account of silt and which fitted the observed data equally well. For Beleida one such formula was: $V = 0{\cdot}02\,R^{0{\cdot}99}S^{1{\cdot}03}$. He understood, however, that Mr. Buckley recognized that fact, and put forward the Beleida formula on the assumption that silt did affect velocity, and not as a proof of that contention, as was clear from the method of arriving at the formula, which assumed *ab initio* that the coefficient varied as the silt-content and was independent of other variables.

The records obtained from the Buckley-Wilson discharge-recorder were also used in support of the theory. The recorder, which had been in Mr. Butcher's charge for the last 3 years, was a most ingenious piece of apparatus, but it had unfortunately proved somewhat unreliable, due to the frequency of electrical breakdowns; and arguments based on its performance must be accepted with caution. The record of the apparatus for the period referred to in the Paper, July to November, 1921, was reproduced in Fig. 1, Plate 3, and it would be noted that over eight periods the lines of " slope " and " error per cent." were shown broken, indicating that the recorder was out of

Mr. Butcher.

order. In particular, the peak in the error curve on the 27th August must be regarded as an instrumental error, as from the original records it was obvious that the instrument was out of order from 6 a.m. to 6 p.m. on that date. Further, the increase in error to 13 per cent. 2 days after the 18th August, could hardly be attributed to any silting action, as the increase in the error occurred suddenly in two stages, and not gradually, as would have been expected from silting action. The following figures, extracted from the original records, made this clear :—

Fig. 4.



RAYAH TEWFIKI.

	a.m.	Error.
20th August	7.00	Zero.
" "	7.30	7½ per cent. in defect
	p.m.	
21st August	4.30	Still 7½ per cent. in defect.
" "	5.00	11·5 per cent. in defect.

Mr. Butcher. The errors were, therefore, almost certainly instrumental. For these general reasons the effect of silt on velocity did not appear to be established. There was, on the other hand, strong and direct evidence that silt in suspension did not materially affect velocity. In the Tewfiki canal, a large channel taking directly from the Nile, a very constant relation between water-level and discharge existed and had remained unchanged for several years. The relation was checked by current-meter discharges. Under such circumstances R and S depended only on the water-level of the canal; and, if silt affected velocity, the same level should be found associated with higher discharges when the water was silt-laden than when it was clear. *Fig. 4* (p. 255) showed the relation between water-level and discharge, as measured by current-meter, for the years 1920–21; discharges with clear and with silt-laden water (August, September, October) being plotted separately. There was no appreciable difference between the level–discharge relation, whether the water was clear or silty; and it appeared, therefore, that, in the Tewfiki canal at least, silt in suspension did not affect velocity. Mr. Buckley called attention to a most interesting subject, on which more experiment was desirable. The weight of evidence, however, did not appear sufficient at the present stage to warrant any alteration in the standard formulas for discharge in open channels, or in the methods of canal-design.

Dr. Chatley. Dr. HERBERT CHATLEY observed that some partial explanation of the variation of discharge with silt might be sought in the changes of viscosity and density, and the ratio of those two quantities, the "kinematic viscosity." According to Osborne Reynolds's rational formula for steady flow, if the index of variation of the velocity with head were n, then the head would also vary as the $(2 - n)$ power of the kinematic viscosity.[1] Since n was the reciprocal of the slope-index, whenever that index was more than 0·5, the viscosity factor came in. In Mr. Buckley's Beleida formula, the slope-index was 0·72, an extraordinarily high value, of which the reciprocal was 1·4. The latter was, so far as he was aware, unprecedentedly low, and suggested semi-viscous conditions. Incidentally, if viscosity must be considered, temperature-effects would also be quite noticeable. Dr. Chatley had endeavoured, without much success, to accumulate data as to the viscosity of suspensions, but in the meantime it might be noted that cement slurry (density, say, 1·2) had been recently reported to have a viscosity of between 8 and 30 water units (British Associated Portland Manufacturers). Up to low concentrations the viscosity was probably a linear function of the concentration,[1] but it must increase very rapidly for densities in silt-suspensions of about 1·5, since it was something like 1,000 million water units for plastic clay.[2] **Dr. Chatley.**

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[1] A. H. Gibson, "Hydraulics and its Applications," pp. 198–200. London 1912.

Dr. Hurst. Dr. H. E. HURST remarked, with regard to the theory put forward by Mr. Buckley, that a certain amount of experimental work on quantities of silt in suspension had been done under Mr. Buckley's direction, but he gave no observations on the thickness, viscosity, and density of the layer in contact with the bottom. Failing that experimental knowledge, its properties were largely hypothetical. Mr. Buckley had examined observations of discharges, levels, slopes, and cross sections, and had found certain cases where departures of velocity in excess were associated with deposition of silt or conditions likely to produce deposition. Formulas of the general type $V = cR^p S^q$ had been adopted to express the relation between velocity V, slope S, and hydraulic radius R. For a given channel c was in some formulas a constant, varying with the type of channel, and in others a function of R and S. The question arose as to how R and S should be measured, for even in a uniform channel like a canal, if silting and erosion took place, R varied from point to point along its course, and so also did S. The variation was much more pronounced in a river like the Nile, where, even at the same point with the same water-level, R varied from time to time. The variations of section at Khannaq in 1920 were shown in *Fig. 5* and also in the Paper. It would thus appear necessary to measure R at several points in the neighbourhood of the discharge-site so as to eliminate the effects of such variations. Slope was measured either (*a*) by the difference of level at two points a considerable distance apart, in which case a good value was obtained for the average slope for that reach of the river, but not necessarily for the slope at the discharge-site; or (*b*) by the difference in level between two points not far apart, but both near to the discharge-site, in which case the difference of level was too small to be measured accurately. The following Tables illustrated the difficulty of measuring slope accurately. They were taken from the records of the Egyptian Irrigation Department. The gauge-readings for Khannaq were made on the gauge described in Appendix B of the Paper. Ten-day mean values had been taken to eliminate, as far as possible, accidental effects.

The distance from the Assuan gauge to the Khannaq gauge was about 28 kilometres. In the first Table the slope on the up-

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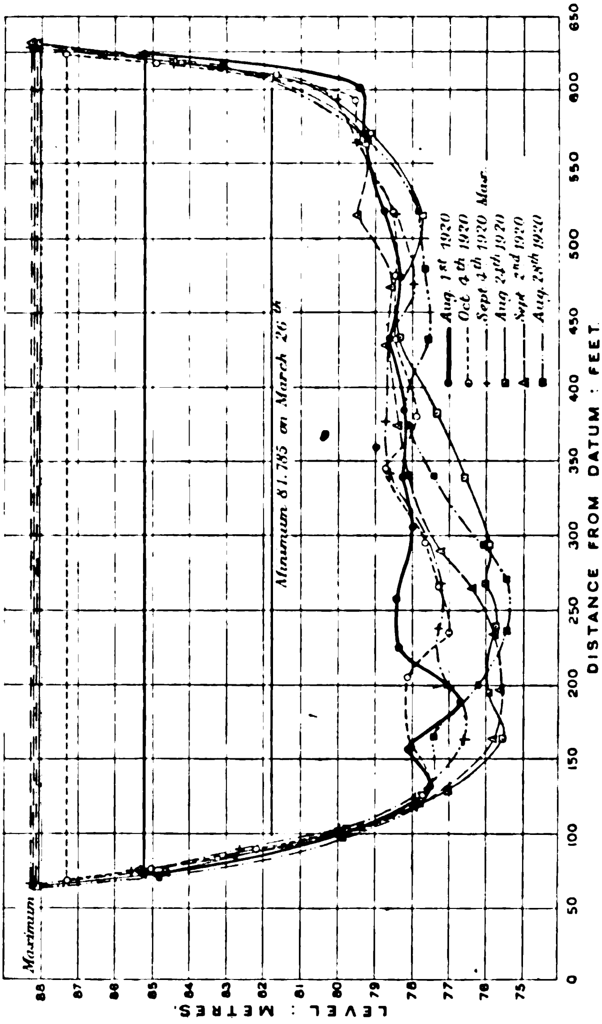
[1] A. Einstein and E. Hatschek. See W. Ostwald's "Handbook of Colloid-Chemistry," 2nd ed., London, 1919.

[2] A. S. E. Ackermann, Trans. Soc. Engineers, 1921, p. 87.

Dr. Hurst. stream section was at the beginning twice that on the down-stream section, but it changed over and became less than half the down-stream slope at the finish. The mean slope over the 3 kilometres ranged from 44 to 75 per cent. of the slope from Assuan.

Fig. 5.



CROSS SECTIONS AT KHANNAQ—1920.

The gauges on the Menufia canal were built on the walls of regulators and were read by the ordinary watchmen. They might be slightly affected by the draw-off of the canals.

Dr. Hurst.

KHANNAQ DISCHARGE-SITE (*Mean Values*).

Period.	Slope over 1½ Kilometre.		Mean.	Slope from Assuan to Khannaq.	Slope over 3 Kilometres as Percentage of Slope from Assuan.
	Up-stream Gauge to Site.	Site to Down-stream Gauge.			
1920.	Centimetres per Kilometre.				
September 2-9 . . .	6·5	3·3	4·9	7·4	66
October 16-25 . . .	5·2	5·1	5·2	6·9	75
October 26-November 4	5·1	4·0	4·6	6·8	68
November 5-14 . . .	4·9	3·8	4·4	6·6	67
November 15-24 . .	4·2	3·7	4·0	6·3	64
November 25 – December 4	3·6	3·6	3·6	6·0	60
1921.					
January 8-17 . . .	1·8	3·7	2·8	5·5	51
January 18-27 . . .	2·0	3·7	2·8	5·6	50
January 28-February 6	1·9	3·7	2·8	5·5	51
February 7-16 . . .	1·8	3·6	2·7	5·5	49
February 17-[illegible]6 . . .	1·5	3·3	2·4	5·4	44

MENUFIA CANAL.

Period.	Slope from Head Regulator to K. 5·9. (1)	Slope from Head Regulator to K. 11·1. (2)	Mean. (3)	Slope from Buckley-Wilson Recorder K. 2 to K. 7 (4)	Percentage Difference of Col. (4) from Col. (3).
1921.	10-Day Means.				
July 1-10	5·4	5·2	5·3	5·2	−2
July 11-20	6·5	6·2	6·4	5·8	−9
July 21-31	6·8	6·5	6·6	6·7	+1
August 1-10	7·4	7·5	7·4	7·7	+4
August 11-20	6·5	6·4	6·4	6·7	+5
August 21-31	4·6	4·8	4·7	5·1	+8
September 1-10 . . .	5·6	5·6	5·6	5·7	+2
September 11-20 . . .	5·9	5·6	5·8	6·6	+14
September 21-30 . . .	5·5	5·1	5·3	5·8	+9
November 1-10 . . .	5·0	4·6	4·8	4·5	−6
November 11-20 . . .	4·9	4·5	4·7	4·4	−6
November 21-30 . . .	4·9	4·5	4·7	4·8	+2
Means	5·8	5·5	5·6	5·8	..

Dr. Hurst. October was omitted, as during most of that month the recorder was not working. The slopes of columns (1) and (2) of the Table referring to the Menufia canal agreed more closely than those of (3) and (4), and even on the means of 10 days the slope measured by the recorder ranged from 9 per cent. in defect to 14 per cent. in excess of that given by the mean of the gauges, although the two estimates referred to almost the same piece of water.

A comparison of single days showed :—

Differences between slopes as given by gauges (Columns 1 and 2).

107	differences	less than	0·5 centimetre per kilometre.
27	,,	between	0·5 and 1·0 centimetre per kilometre.
1	difference	of	1·0 centimetre per kilometre.

Differences between the recorder slope and the mean slope from the head-regulator to the gauges (Columns 3 and 4):—

94	differences	less than	0·5	centimetre per kilometre.
21	,,	between	0·5 and 1·0	centimetre per kilometre.
16	,,	,,	1·0 ,, 1·5	,, ,,
2	,,	,,	1·5 ,, 2·0	centimetres ,,
2	,,	,,	2·0 ,, 2·5	,, ,,

Thus, in 20 cases out of 135 the slope as given by the recorder differed from the mean slope given by the gauges by more than 1 centimetre per kilometre. As the mean slope was about 5·6 centimetres, 1 centimetre represented 18 per cent. The maximum difference was 45 per cent. The error on the discharge was about two-fifths of the error on the slope.

Examining some of the details of Mr. Buckley's argument, the large errors of the recorder might be divided into three groups :—

(1) The period 20 August–15 September.
(2) ,, ,, 15 September–30 September.
(3) ,, ,, 22 November–3 December.

In the first period an error, which reached 13 per cent. and then decreased, coincided with a drop in slope followed by a rise. That was the evidence from the recorder for the silt theory. There were, however, several other considerable changes of slope which occurred while the water still contained a high content of silt. Those were not discussed. In the second period errors of the recorder occurred up to 10 per cent. in the opposite direction, which were said not to influence the main argument. The reason for their dismissal given in Appendix A was that "with the conditions of heavily-fluctuating levels . . . the recorder and official gaugings are not strictly comparable." Such would apply with even more force to the period from the 20th August

Dr. Hurst.

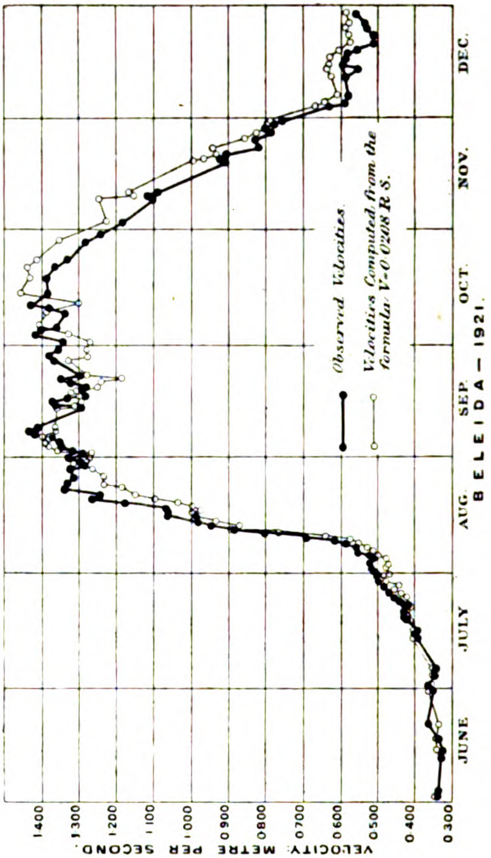
to the 15th September, which was relied upon as evidence for the silt theory, for, counting only changes of 20 centimetres or more between one day and the next, the first period contained changes of gauge, at the head of the reach, of 20, 27, 23, 21, and 22 centimetres, while the dismissed period contained a single one of 24 centimetres. In the third period the error reached 10 per cent., and that was explained as due to the slope-cam not being correct at the low slopes of 4-5 centimetres per kilometre, and to the fact that its correction should be 3 per cent. at 5 centimetres slope, and 10 per cent. at 4 centimetres slope. Fig. 1, Plate 3, showed that from the 8th to the 23rd November the mean slope was 4·4 or 4·5 centimetres per kilometre, and the mean error was zero, while from the 23rd November to the 2nd December the mean slope was about 5 centimetres per kilometre, and the mean error was 5 or 6 per cent. The proposed correction, therefore, would not remove the errors. The discussion of the errors of the recorder given in the Paper produced no evidence as to any effects on velocity due to the deposition of silt.

In regard to Baqlawis in 1919 the state of the river from the 12th to the 14th October was selected for examination because of the drop in velocity and change of bed. But the velocity was falling most of the time from the 19th September, and the bed was also changing in an irregular manner (Fig. 3, Plate 3). The same method of dealing with particular instances was followed throughout the Paper; and in view of the uncertainties in slope-measurements already shown and the variability of the river-bed, the method was quite incapable of leading to reliable conclusions. It was useless to cite a few variations of 8 to 21 per cent. in velocity in support of a theory, when the estimate of the slope over 3 kilometres at a discharge-site might vary from 45 to 74 per cent. of the slope over 28 kilometres, which was used in calculation. The only possible way of dealing with such data was by the recognized methods of statistical science, taking proper account of all the observations.

The question of formulas to represent velocity in terms of slope and hydraulic mean depth was interesting. All such formulas were empirical, and many could be found to fit given sets of observations. Mr. Buckley showed (Fig. 11, Plate 3) that those of Bazin, Kutter, and Manning did not increase rapidly enough with R and S to cover the range of conditions of the Nile from low stage to flood which, as could be seen from the Figures, was extreme. Mr. Buckley stated that that was due to the fact that silt had not been taken into account, and he produced the formulas containing the term silt-content, Z, which fitted the facts at Beleida and Khannaq The same result could be achieved without using silt-content at all

Dr. Hurst and in fact the very simple formula $V = 0{\cdot}0218\ R\ S$ had been found by Dr. Hurst's colleague, Dr. P. Phillips, which fitted the facts at Khannaq for 1921 as well as the more complicated one. The fact that the silt in suspension was closely correlated with the hydraulic radius (Fig. 10, Plate 4) necessarily implied that, if V was a function of R, it could also be represented by an equation into which silt-content entered. Dr. Phillips had found that the equation $V = cRS$ with different constants fitted the Main Nile at Beleida and Khannaq and the Blue Nile just above and at 450 kilometres from its junction with the White Nile. Both those rivers were silt-bearing in flood. The same formula also expressed the velocity for the White Nile at Malakal, in which there was very little silt. Some of the results were shown in *Figs. 6, 7,* and *8.* S in all the

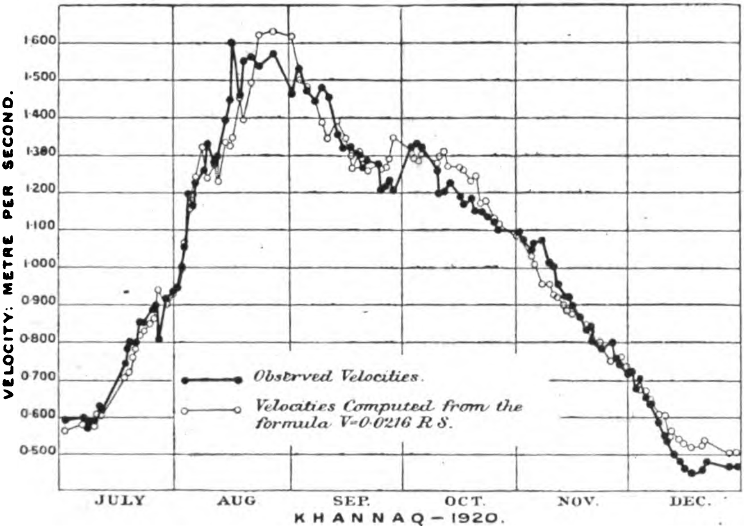
Fig. 6.



Dr. Hurst.

formulas denoted an average slope over a comparatively long distance, and so might depart from the slope at the place of observation of discharge. No significance could be attached to the exact value of the constant in the equation, and accidental errors would probably be fairly large. The fact that the formula $V = cRS$ fitted a non-silty as well as silty rivers robbed the Beleida formula of any significance. The results derived from fitting the formula $V = cRS$ to various stations might be used to examine the probable truth of the silt theory. According to the argument of the Paper, if there was

Fig. 7.



	Year.	Rising River.	Falling River.
	Main Nile.		
Khannaq	1920	$V_0 = V_c$	$V_0 = V_c$
Khannaq	1921	$V_0 < V_c$	$V_0 = V_c$
Beleida.	1921	$V_0 > V_c$	$V_0 < V_c$
	Blue Nile.		
Soba	1921	$V_0 = V_c$	$V_0 > V_c$
Wad-el-Aies	1922	$V_0 = V_c$	$V_0 < V_c$

Dr. Hurst. silt in the water, with a falling velocity, for the same values of *RS*, the velocity was higher than with a rising velocity. Hence in the formula $V = cRS$, which took no account of whether the velocity was rising or falling, the observed velocities (V_0) should in general be greater than the calculated (V_0) during the falling period and less than the calculated during the rising period. Of course the uncertainties in *R* and *S* already mentioned would cause accidental variations. The foregoing were average results, omitting the period when silt was negligible, and also the crest of the flood when the river fluctuated considerably.

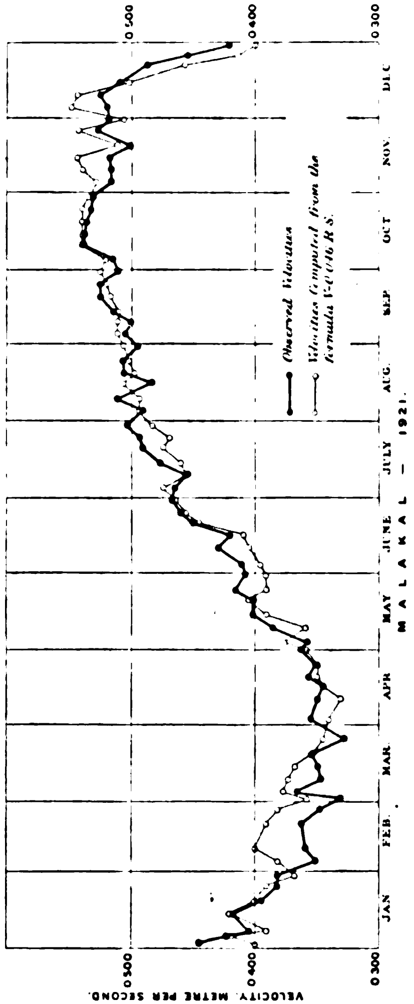


Fig. 8.

There was nothing systematic in the variations tabulated. To sum up the argument of this discussion, the large variations in the observed quantities *V*, *R*, and *S*, and the errors involved in measuring *S*, made any other than a statistical investigation, by the recognized methods, useless; and therefore the arguments of the Paper proved nothing. The evidence of the foregoing Table, however, rendered it unlikely that the complete statistical examination would show any large effects on velocity due to the deposition of silt.

Mr. Kanthack Mr. F. E. Kanthack observed that Mr. Buckley's Paper was a valuable contribution to the study of the flow of silt-laden water in rivers and canals, and the Author had developed an interesting theory to account for fluctuations in the mean velocity in the Nile

Mr. Kanthack.

and canals taking off therefrom. He rightly abstained, however, from making any sweeping deductions from the Nile experiments. There were obvious limitations to the decrease in the coefficient of roughness resulting from the deposition of a silt-film on the periphery of the channel and to the consequent increase in the mean velocity. In the first place, the formation of such a lubricating lining must depend largely upon the character of the silt carried in suspension and also upon the character of the heavy rolling sand and silt, if such material were present. It could not be assumed that an increase in the quantity of silt in suspension in water flowing in an open channel automatically caused an increase in the mean velocity, and consequently in the discharge, for a given gauge-reading. The contrary had been proved in the case of other rivers. He called Mr. Buckley's attention to the researches by Swiss engineers, which were quoted in the "*Annales Suisses d'Hydrographie.*"[1] The information was contained in a comprehensive memorandum by Dr. Léon W. Collet on "*Le charriage des alluvions dans certains cours d'eau de la Suisse.*" It was found that in the case of Swiss rivers the mean velocity was reduced very considerably with the increase of solid matter in suspension, the lowering of the velocity being attributed to the increased viscosity of the water due to the heavy charge of silt derived from schistose rocks. The diminution in the velocity appeared to have been observed mainly in the surface velocities. That an increase in the quantity of silt carried in suspension caused retardation in the velocity of flow had been demonstrated by French and Swiss engineers more than half a century ago. The theory of the lubricating film of "slurry" would appear to be tenable only if the periphery became covered with mud of a fine character. In most cases, however, the character of the solids moving along the bed of a stream or canal, during periods when the water was heavily charged with silt, was not such as would provide a lubricating blanket. The coarse sand and silt rolled along the beds of South African rivers, for example, was more likely to move along in a series of waves or corrugations, the effect of which must be to stimulate eddies and increase the vertical component of the velocity of movement in any vertical plane rather than to diminish it. Beyond the fact that it was present in embarrassing quantities, very little was known about the quantity and behaviour of rolling sand and silt in a canal or river. It was obvious, however, that in so far as the coefficient of roughness was concerned, the character of that material must be, in most cases, the predominating factor, and not the

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[1] Vol. ii, Berne, 1916.

Mr. Kanthack. quantity of silt in suspension. In so far as the silt in suspension affected the issue, the character thereof from a mechanical point of view must obviously be of great importance. It hardly seemed possible, therefore, that an expression, such as the Beleida formula, could be of general application. The silt factor, if taken into account, must embrace many factors besides the quantity of silt in suspension. It was obvious also that the mean velocity could not go on increasing proportionately to the quantity of silt in suspension or transported by the river. On the supposition that silt of a certain kind would form a lubricating layer all over the wetted perimeter, such an effect must clearly reach its maximum with water carrying a certain percentage of silt of the right kind. The coefficient of roughness would gradually diminish till that maximum effect was reached, after which the coefficient should become more or less stationary. Meanwhile the retarding effect of excessive concentration of silt carried by the water was likely to come into action in an ever-increasing degree. There were so many diverse factors coming into play in such cases that it seems hopeless to attempt to establish a general formula for the velocity, bringing in the silt factor. What was required in the first instance was a series of discharge-curves for the same river or canal, at the same place, relating to different conditions of silt-content of the water. The assumptions regarding the flow of muddy water in open channels were not corroborated by his practical experience of the regulation of a large canal in the Punjab during the period when the silt-content of the water was at its maximum, and full supplies were required in all branches and distributaries throughout the system. He had been frequently puzzled by unaccountable deficiencies of supply delivered at distant points, which were vaguely attributed to little-understood fluctuations in the absorption losses. It was more likely that the large deficiencies were due to the varying concentration of silt in the water. He had never found that an excess of water was being delivered to the canal-system during the silt season, which should have been the case, according to Mr. Buckley's theory.

Mr. Lacey. Mr. J. M. LACEY asked whether Mr. Rothery could produce evidence in support of his statement that at some period of time the Colorado river oscillated between the Gulf and the Salton basin; and also whether he could say what had caused the barrier between those two features. Before 1909 the river appeared to have taken an almost direct course to the Gulf, and since that date it had altered its course, always trending towards the right, until now it ran on the crest of the barrier, and was only kept from falling into the Salton Sea by artificial means. Some force must be acting on the river to move its course up an incline. The river fell some 12,000 feet from its source in the Rocky Mountains to the coastal plains in a distance of about 900 miles, as measured on a map, cutting the Grand Cañon through the arid plateau of Arizona. The debris brought down by the melting snows, instead of accumulating in screes on the river-margin, was swept away through the cañon, so that the quantity of silt carried down and deposited where the velocity of the current slackened must be considerable. It was possible that the deposit caused the outfall of the river into the gulf to deteriorate. Improving the outfall by widening and dredging, and suitable training-work, might tempt the river back to its original course. While in charge of river-conservancy operations connected with the Godavery and the Kistna rivers, he had always insisted on the borrow-pits being placed on the river side of the flood-banks to ensure their silting up. It frequently happened that these rivers came down in flood when the deltas were dry and when the soil was full of large cracks; if the borrow-pits were on the countryside, and a crack extended through the soil into the borrow-pit, the head producing flow through the crack was increased. He considered that borrow-pits should not be continuous, and had found that sufficient care was rarely taken in their location. Scours of over 20 feet were not uncommon in Indian rivers, and the difficulty of constructing a permanent structure at Yuma should not be insurmountable. A dam 500 feet in height seemed a heroic remedy. A maximum flood-discharge of 220,000 cusecs for a catchment-basin of 242,000 square miles, or less than 1 cusec per square mile, was not excessive, compared with Indian rivers. A smaller dam with improved outfall of the river might meet the case. The rate of silting up of the reservoir was a factor that would have to be taken into consideration. **Mr. Lacey.**

Mr. Buckley was to be congratulated on the meticulous care he has taken to arrive at facts. The conditions described were those of a placid river under control issuing from a reservoir where the heavier material which was rolled along the bed was deposited, so that the silt transported was held in suspension, and of the lighter kind, consisting mainly of fine particles of alluvial matter. The conditions appeared to be similar to those of the Godavery Western Delta canal-system, for there, owing to the existence of deep water in the river in front of the head sluice of the main canal, only the fine particles of alluvial matter which rendered the water turbid were carried into the canal-system. The difficulty in that system was to prevent bed- and berm-scour due to excessive velocity, and to prevent deposition if the velocity was low. The critical velocity

Mr. Lacey. to prevent deposition had been found to be $V = 0{\cdot}67d^{0{\cdot}55}$; where v denoted the mean velocity in feet per second, and d the depth of the canal in feet. The limiting mean velocity to prevent erosion had been found to be 2·38 feet per second. For small channels the critical velocity should be increased.

Mr. Lindley. Mr. E. S. Lindley considered that the importance of the problem with which Mr. Buckley's Paper dealt could hardly be exaggerated. Though the Author stated that the effect which he now reduced to figures had been known for some time, and referred to one quite specific statement of it, it was significant that the able and exhaustive investigation reported in Part III of the Technical Reports of the Miami Conservancy District ("Calculation of Flow in Open Channels," by Mr. Ivan E. Houk), did not refer to it.

In irrigation work in the Punjab there had long been a suspicion that the velocity of water in rivers and canals was affected by the silt in suspension; and more recently, definite observations had begun to crystallize the suspicion into suggestion. The Punjab, however, was still very far behind the stage to which the question had been brought by Mr. Buckley's Paper. The effects reported were fully borne out by Punjab experience; and he proposed to discuss the reasons suggested for these effects, after presenting such data as were available.

An anonymous writer in "Indian Engineering"[1] stated, in 1892, that the resolved component of velocity in a vertical direction alone made mechanical suspension of silt in flowing water possible. He called attention to the fact that in open channels floating scum and dust often collected from the sides into a central streak, and explained this by spiral currents, the components of which, perpendicular to the main flow, were upward at the sides and downward at the centre, and therefore inward on the surface and outward on the bed; and he stated that in suitable lights he had been able to distinguish the flattened cylinders of more silty water from the central zone of clearer water under the streak of dust and scum. He claimed that this form of flow always indicated a channel in which silting was not going on. Referring to stream-line flow, he wrote: "Such flow will give a greater discharge for a given surface fall than if there were, in addition, motion in a transverse plane, which would also absorb energy but could not affect the discharge; but, on the other hand, the silt-carrying capacity of such a stream would be very small or absolutely nil." This anonymous writer made about 3,800 observations on seven channels, selected as taking off from the canal under nearly similar conditions. Plotting Kutter's *n* against the sectional area of silt deposited per cusec of discharge, he showed that *n* decreased with an increase of silt-deposit. "It follows therefore," he said, "that all additional motion in a transverse plane—which has no effect on the discharge and which must necessarily be accompanied by friction,[1] and in consequence increase the surface fall—will increase the value of *n* without any alteration in bed and sides of the channel." He stated that plotting the sectional area of silt deposited against the ratio of breadth to depth, showed at least that it was neither the surface fall nor the mean longitudinal velocity that determined the silt-carrying capacity, but rather the ratio of breadth to depth. Six out of the seven channels gave points lying very close to one curve; the seventh, with a relatively lower value of *n*, showed "free ends of vortices on the surface." Mr. Lindley.

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[1] Vol. xii. (Reprinted in Punjab Irrigation Branch Paper No. 7.)

The foregoing implied that the carriage of silt used up more of the energy due to the fall in the channel, and reduced the discharge, which was the opposite to the effect described in Mr. Buckley's Paper.

In 1914 Mr. Lindley was in charge of the headworks of the Lower Jhelum Canal at Rasul; the head-regulator of the canal at that time contained narrow wooden gates, which could be dropped to close the openings, while for regulation two were slung on travelling crabs and raised or lowered as required. The canal indent was given on a gauge 14 miles down the canal, but regulation was done by maintaining a steady level on a gauge immediately below the regulator, on one flank, and practically at the head of the short reach in which the width was reduced from that of the regulator to that of the canal. After every flood or freshet those in charge of the lower channels complained of having been given excess supply, while those in charge of the headworks rejoined that the head gauge had not been allowed to exceed. In the second flood during his incumbency, he confirmed the statements that an excess had really been passed down, that the head gauge had not been exceeded, and that a gauge 1,000 feet lower down in the parallel canal section had also remained steady. Silt was not suggested as a cause, and the only suggestions arrived at were that when head-regulator gates were closed from the two flanks, the level at the gauge on one flank might be lower than the average water-level; or that the higher velocity through the narrower total opening persisted in the canal. Though this did not seem to account for the amount

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[1] *Sic*, query "loss."—E. S. L.

Mr. Lindley. of the excess, there was no time for theorizing. He therefore laid down that in the next flood the water-level at the gauge should be reduced below indent by 0·05 foot for every regulator-gate dropped. Not only did this rough rule avoid excesses in subsequent floods, but it proved almost exact in keeping the supply steady. The canal supply was up to 4,000 cusecs, which meant a gauge of about 5·8 feet; in flood up to ten gates might be dropped before it was judged necessary to close the canal altogether because of silt in the river, so that under the rule referred to the gauge would be reduced to 5·3 feet, equivalent to a reduction of discharge of 500 cusecs. On one occasion he kept the canal steady with the regulator fully open, and had water-levels marked at intervals down the canal; water-levels were then noted again while eight or ten gates of the regulator were closed and the corresponding reduction of gauge was maintained. He believed that the river levels were raised by closing the under-sluices to the necessary degree, and without occurrence of any flood: the difference between the two water-levels decreased with distance down the canal, till at about 10 miles the difference became constant and about 0·1 foot, indicating a small reduction of supply.

The head-regulator had now been remodelled, having wider openings fitted only with rising gates, which were all kept level. A reduction of gauge was now made of 0·1 foot for every 0·3 foot that the river rose. With the former fitting of the regulator the closure of two gates would correspond roughly to a rise of 0·3 foot in the river; so that from this aspect the two rules were the same.

No one had suggested that the flushes in the canal at Rasul, when the head gauge was kept steady, were due to any effect of silt in the water; in fact, unless Mr. Lindley's recollection of his test for fixing water-levels was at fault, such flushes did not depend on the occurrence of a silty flood in the river. Investigation of the phenomenon was difficult, because floods were there of short duration; and where many square miles of populous country were absolutely dependent, even for their drinking-water, on a canal, it was not possible to do more than a minimum of experimental regulation that might produce undesirable results.

Similar flushes had been noticed at Balloke, the headworks of the Lower Bari Doab Canal, and investigated by the Executive Engineer, Mr. E. S. Crump. The conditions on that canal differed from those at Rasul and lent themselves better to investigation. Instead of the usual fixed weir with falling shutters on the crest, there were Stoney sluices across the whole river, which were closed to the extent necessary to pass the supply into the canal, and, as the river

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rose in flood, the sluices were opened to maintain that level, until the river perhaps rose still higher with all gates fully open. The head-regulator was fitted with rising sills whose top-level was practically at the full-supply level of the canal; the regulator openings were closed by gates lowered each by its own standing gear. The head-works were much farther from the hills than were those at Rasul, and the silt carried by the river was much finer: in the first 5 miles there was a slight deposit of silt on the canal-bed, but in the next 7 miles the original clay bed was kept scoured clean. A report for the year in which the method of avoiding these flushes was discovered gave some figures. In the first two river freshets, a recording gauge showed that water-level in the canal below the head-regulator had not been allowed to exceed indent; yet a gauge 12 miles down the canal rose by an amount corresponding to 300 to 400 cusecs excess on a discharge of 7,000 cusecs. There was no rain to account for such flushes; and as regarded an alternative suggestion, namely, that the rise of water-levels in the river decreased the percolation flow from the canal which followed it fairly closely for some miles, the flush was much too prompt, and exceeded the total absorption loss above the first point at which an increase of gauge was apparent. On the arrival of the next flood, the measure was tried of regulating entirely by keeping the river-level above the barrage steady, leaving the regulator-gates untouched and noting the resulting gauges: the canal head-gauge fell; a gauge 6 miles down the canal fell nearly as much; no other gauge was noted above 25 miles, where a gauge above a canal fall showed that the discharge had been nearly steady; other gauges farther down the canal gave the same indication. While with only 8,000 cusecs in the river, an observed discharge of 6,400 cusecs in the canal had given a gauge of 6·15 at the head a month before, now, with 23,000 cusecs in the river, an observed canal-discharge of 6,850 cusecs gave a gauge of only 6·19. This method of regulating was therefore now adopted during floods, since the canal gauges were shown to be affected for a greater distance from the head than such as rendered them of any use for regulation.

It was then suggested that the flushes might be due to scour of the silted bed, though it seemed unlikely that scour of several miles could take place so rapidly, and the gauge at the sixth mile showed variation below the reach in which the bed was silted; but soundings taken 2½ miles from the head, at a site known to be silted, showed no perceptible change of bed. It was therefore proved that an unaltered channel flattened its slope; and that the coefficient of rugosity was reduced.

Mr. Lindley. Mr. Crump's tentative physical explanation was that the very fine particles in suspension in flood-water formed the nuclei of extremely minute globules of water, and that ordinarily visible eddies led to smaller and at last molecular eddies, in which the conversion of energy into heat took place; further, that the globules of water around silt particles were held, and as the usual molecular eddies therefore could not take place, the ordinary dispersion of energy was decreased, and less slope was needed to cause the same velocity.

Turning to some data of a totally different kind, in October, 1920, Mr. Lindley made a hydraulic survey of the Jhang Branch Lower of the Lower Chenab Canal, taking special care to include in it a careful estimate of discharges. From these and from the average dimensions of ten reaches, into which the 38 miles of channel naturally divided, he calculated and plotted the various factors. The range of data ($n = 0{\cdot}020$–$0{\cdot}0285$ and critical velocity ratio $= 1{\cdot}04$–$0{\cdot}84$) was not wide enough to show a clear law, with the natural scatter of the plotted points, but it was quite distinct that a low n connoted a high critical velocity ratio (c.v.r.) and vice versa. At the same time Mr. Crump made a similar survey of 33 miles of the main line of the Lower Bari Doab Canal. He took his sections in greater detail, and calculated the factors at every thousand feet; he thus obtained more points over a wider range ($n = 0{\cdot}012$–$0{\cdot}035$, and c.v.r. $= 1{\cdot}38$–$0{\cdot}60$) from minimum to maximum, than the averages of Mr. Lindley's survey. These points quite distinctly showed n inversely proportional to the critical velocity ratio.

In commenting on this report, Mr. F. R. Burkitt, using the Manning formula as giving results sufficiently close to that of Kutter, ignoring the slight difference between the exponents in that formula and the Kennedy formula, and applying a value of n suitable to a river, and the actual slope, derived a value for the c.v.r., which enabled him to calculate depths of scour. From this it became obvious that n and the c.v.r. calculated from any data whatever must be in inverse proportion as long as the slope was the same. But as Mr. Crump's data showed n as varying from $0{\cdot}015$ to $0{\cdot}035$ on the same channel, without ostensible reason, the method was of doubtful utility. Going back to the data themselves, Mr. Lindley found that the velocity varied as the power $1{\cdot}42$ of the hydraulic mean radius.

Data which would afford Mr. Buckley the means of testing almost directly the applicability of his Beleida formula were to be found in the published reports of the Indus River Commission. Since 1902 discharge observations had been made regularly of the Indus at several points, and silt samples had been collected. Some, at

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least, of the plotted curves of Kutter's n showed regularly higher values in winter when the water was clear, and lower values in summer when floods brought down silt. Cursory comparison of some of the individual curves with the curves of silt-content showed that their variations were interdependent, but Mr Lindley had not found time for the elaborate analysis necessary to ascertain the degree of interdependence and the law underlying it. A rough indication of the amount of variation might be given from the data for the Kotri site for the years 1911–15. During the winter n ranged between 0·018 and 0·028, with 300 to 500 grains of silt per cubic foot of water; during the summer n ranged from 0·006 to 0·011, with 1,300 to 1,800 grains of silt per cubic foot of water. The silt was estimated by filtering a measured sample of water, drying, and weighing. Before entering upon a laborious investigation of the data from any of these sites, its peculiarities should be noted, such as the positions of the gauges at which water-levels were taken for the slope element in calculation.

Being entrusted with the collation of research work done in the Punjab Irrigation Department, Mr. Lindley had endeavoured to collect information on the question raised by Mr. Buckley, but so far only the two following communications had reached him.

Lalla Bur Singh had reported that while a party of engineers were making frequent discharge observations of the Abohar Branch of the Sirhind Canal, his duty being to see that the discharge at the head was kept constant and to observe it at the first site, he noticed that with a steady gauge the discharge varied, and became greater the more turbid the water. The discharge varied by as much as 5 per cent., and while the regulating gauge was steady, gauges lower down showed these variations. Mr. P. Claxton had called attention to an article contributed by him to "Indian Engineering."[1] At one place he said that when the load of silt increased the bed became rougher; this was what would, *prima facie*, be expected, meaning that an increase of silt-charge resulted in a retardation of velocity; similarly it would, *prima facie*, be expected that the transportation of an increased silt-charge used up more of the energy given by the slope or fall of the channel, leaving less for the maintenance of velocity. The primary purpose of Mr. Claxton's article being to call attention to the formation and destruction of eddies as the essential study in flowing water, he said[2]: "Consider a stream with any silt-ratio; some of the energy of eddies is absorbed in supporting silt, and therefore a proportion only remains for

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[1] Vol. lxvi, 1919, pp. 39 *et seq.*

[2] *Ibid.*, p 154.

Mr Lindley. resisting acceleration. If therefore there were no silt, and the nature of the bed could remain the same, the velocity should decrease because of the greater available resistance of eddies." In other words, a certain channel-bed would, because of its form, throw off a certain amount of flow more or less contrary to the main flow; the more of this contrary flow that was used up in supporting silt particles that were trying to fall, the less was left over for destroying the energy of the main flow.

But the most pertinent observation of Mr. Claxton's communication was the following quotation from Mr. G. K. Gilbert's report of his study of the transportation of debris by running water :–

> "When the load of silt is small, the bed is moulded into ridges or hills called dunes, which travel down-stream as the current erodes their up-stream faces and deposits material on their down-stream faces. With progressive change of condition tending to increase the load, the dunes gradually disappear, and the channel becomes smooth or even. With further changes the bed again becomes uneven, and a system of dunes will form that appear to travel up-stream owing to erosion on the down-stream faces and deposition on the up-stream faces. Both rhythms of débris movement are initiated by rhythms of water-movement."

While it was known that flushes similar to those reported from Rasul and Balloke had been observed at other headworks, and that at some of them regulation was corrected by rules of thumb, only one report had been received that was worth quoting. Rupar was the headworks of the Sirhind Canal, on the Sutlej, relatively much nearer to the hills than Rasul: silt-control was there affected by using the pond above the headworks as a settling-pond and escaping only over the weir while the canal was opened. When the silt deposit rose to a level that was likely to lead to excessive admission of silt to the canal, the canal was closed and the under-sluices were opened to scour it away. Mr. J. L. Roy reported that, when the time for a scouring approached, a given indent resulted in excess supply in the branches of the canal, as compared with times when the bed of the silt-pocket was lower, and, presumably, rather less silt was entering.

At the headworks of the Sidhnai Canal, the River Ravi had an unnaturally straight reach several miles in length, the "Sidhnai." Observations of river-discharge at this site were therefore likely to be more accurate than at ordinary sites in the determination of surface slope and calculations depending on this. While no observations had been made of the amount of silt in suspension, as long ago as 1893 Mr. Kates suggested that n was lower when the water was silty, and that the bed consisted partly of moving and partly of "wet" silt. Mr. E. S. Bellasis noted that this agreed with his own experience, but Mr. (now Sir) John Benton disagreed. The controversy, beginning with Sir John Ottley in 1890, was not followed up after 1897. **Mr. Lindley.**

It remained to examine the facts brought forward, and attempt to arrive at a physical explanation. With regard to the anonymous writer in "Indian Engineering," the hypothesis that the more flow there was in a plane transverse to the direction of main flow the more silt could be supported, seemed unexceptionable; but it did not follow that, if the silt-charge were reduced, the transverse flow would diminish, leaving more of the energy due to slope to give purely forward velocity. There was a common impression that an increase of silt-charge entailed a roughening of the silted bed by growth of the silt-waves or dunes travelling down it. Presumably the silt-charges and velocities occurring in practice came within the range in which Mr. Gilbert had observed that, with an increase of silt-charge, the dunes smoothed out: in any case, the fact remained that a body of general observation supported Mr. Buckley's observations and what was stated by Mr. Flamant, which was the opposite effect to that to be expected from such a cause.

Referring to the flushes observed at various headworks, at Balloke, at least, other possibilities had been excluded, and the observations there confirmed the Flamant statement of the matter. It was significant that, while most of the observations, including Mr. Buckley's, were of flood-water carrying mud, at Rupar Mr. Roy noted the effect when no more occurred than a trickle of coarser silt over the regulator and on the bed. Similarly, in Mr. Lindley's test at Rasul, the water did not change, but the more concentrated intake might have drawn in more of the lower layers of the smooth flow in the under-sluice pocket, which otherwise carried the heavier silt under the undersluices and down the river; in this test the effect was very slightly, if at all, less marked than when the flow was of water carrying mud and (presumably) still more silt.

In connection with the data taken from hydraulic surveys, it was first necessary to remember that *n* and the critical velocity ratio, calculated from the same data by closely similar formulas, must be interdependent; that the flow was not uniform; that each short reach did not use up just the energy due to the fall in its length, but that the slower deeper reaches used up some of the velocity generated in shallower swifter reaches; and that the level-observations rather tended to smooth out the gradient. But the essential fact remained that where, according to other observations, it was to be expected that the current was maintaining more silt

Mr. Lindley. in suspension, the velocity was greater than would normally be given by the other factors, depth and slope, and vice versa. This meant that the channel consisted of alternating pools and rapids such as were patent on any shallow torrent. Probably every engineer who had had to regulate supplies of silt-laden water in earthen channels had on occasion found his regulation upset, and had even observed other effects that could only be caused by a silt wave, perhaps thousands of feet long and several tenths of a foot high, travelling down the canal and slipping over falls. It was significant that these observations, too, involved no change of muddiness of the water, but only the picking up of some silt that probably travelled only in the lower layers of the stream, by saltation.

Taking the data brought forward by Mr. Buckley, accepting the truth of the Flamant statement, and choosing what was significant in arriving at a physical explanation, the following points emerged :—

(*a*) On the 18th August the actual discharge of the Menufia Canal began to increase relative to the discharge normally dependent only on water-level and slope : this occurred in the middle of a period of reducing discharge and slope. When the silt-charge had increased nearly to a maximum it was suggested that this must therefore have been a time of beginning of silt-precipitation. On the 22nd August the difference of discharges reached a maximum, and on the 27th it began to decrease : on the 22nd the discharge and slope began to increase, and it was suggested that this resulted in silt again being picked up. In support of this suggestion it was pointed out that the silt record of the canal, begun, however, only on the 23rd, showed a peak on the 27th ; the silt record of the Nile showed no corresponding peak, but was rising to its maximum till the 24th, and did not seem to have been observed again till the 31st.

(*b*) On the 13th October, on a falling river, the filling of the Assuan reservoir was begun by closing sluices : the gauge-area curve at Baqlawis showed that on the 14th the river trough began to scour ; from the 12th to 14th the observed mean velocity at that site decreased to an extent that aroused attention. Examining the data of observations, it was found that, across the river-channel, increases and decreases of depth from the 12th to 14th very nearly connoted decreases and increases of the velocity measured at 1 metre from the bed :

also, with neglible change of slope and only 2 per cent. reduction of hydraulic radius, the velocity had fallen 11 per cent.; and further, while the maximum velocities in the section on the two dates were in the ratio 0·91, the peripheral velocities were in the ratio 0·83. Mr. Lindley.

(*c*) On the 16th September the gauge-level at Khannaq on the falling river was the same as on the rising river of the 16th August; the hydraulic radius, sectional area, and slope were also the same on these dates: but the velocity had decreased from 1·48 to 1·32, and the silt-charge had decreased by half.

(*d*) Between the 17th and the 19th October, a closing of the Assuan sluices coincided with a fall of velocity as noticed the year before at Baqlawis, but exhibited much more by the manner in which the Khannaq data had been plotted: the curve of silt-observations showed no effect from this closing of sluices. On the 26th July the gradual opening of the Assuan sluices was interrupted, apparently on a pause in the rise of the river; it was suggested that this resulted in the dropping of silt in the reservoir. The data showed that the river trough immediately scoured and then again silted to normal; and that, in spite of the corresponding increase and decrease of hydraulic radius and an accompanying increase of slope, the velocity decreased and recovered. Altogether, marked alterations of regulation at Assuan produced immediate and remarkable responses in the regime of the river lower down; they seemed to justify the suggestion that any checking of velocity that caused silt-deposition in the reservoir was at once made up by scour below it.

(*e*) Observations at Beleida showed velocities in autumn as much as 50 per cent. higher than those calculated from observed data, by accepted formulas, with coefficients that were correct for the rest of the year: the formula, deduced from these observations and taking into account the amount of silt in suspension, fitted them within about 6 per cent. The test of the formula by the data of a silt-season at Khannaq was also satisfactory.

Of the conclusions arrived at by Mr. Buckley, the most concrete was the Beleida formula. Due caution was shown in discussing the likelihood of its application elsewhere, under different conditions; relevant to this point were the tests already referred to on p. 270;

Mr. Lindley. and what was observed at Rupar (p. 274); in both these cases there was a probability that there was little change of silt-content except in the layers near the bed, and the Rasul test apparently showed nearly as much effect as if more fully silted water had been flowing. In fact, as Mr. Buckley admitted, his quantitative tests being all made under similar conditions, the true cause of the phenomenon might elsewhere not have the same relation to the total silt-content as it had there.

Mr. Buckley suggested that the relative increase of velocity occurred only when silt was actually being precipitated; that during silting the first step was the formation of a blanket of slurry flowing on the bed, which might solidify if not scoured away before it did so (p. 201, Appendix A); that not only did removal by scour of this blanket of slurry terminate the increase of velocity, but that the silt picked up by scour did not again form such a blanket. The last conclusion seemed to rest principally on what was noted in clauses (*b*) and (*d*) above, and did not, *prima facie,* seem to be entirely in accord with Mr. Lindley's two observations again referred to in the preceding paragraph; but in these cases an addition of silt was made to water that was flowing in the canal in a state of balance between silting and scouring; while in Mr. Buckley's cases referred to, he suggested that that state of balance was disturbed by deposition of part of the balanced silt-charge above Assuan, that this deposit was being made good by scour below Assuan. Presumably at the site of observation the deposit had not yet been made good, and the silt taken up had all to go into suspension short of the degree at which the blanket of slurry formed. Mr. Lindley could not find in his data any fact that tended to disprove these conclusions arrived at by Mr. Buckley. The flushes experienced in canals at Rasul, Balloke, and Rupar accorded with Mr. Buckley's hypothesis of a "blanket of slurry" only if the heavy silt could drop and form that blanket in a relatively short distance after the complete mixing and disturbance that took place in falling over the sill of the head-regulator.

But the corollary to these conclusions seemed to be that the coefficient of velocity was a function not of the amount of silt in suspension but of the rate of increase of silt; perhaps not even of the absolute rate of increase of silt-charge, but of the rate of increase in excess of what was "absorbed" by actual solid silting under the blanket of slurry. If this were true, a general formula would be complicated and quite impracticable; but as the Beleida formula appeared to fit the Nile, and the amount of silt determined by the method followed there, similar formulas good enough for other conditions should be derivable. In this connection Mr. Lindley

Mr. Lindley. suggested that it would be of interest to test the application of the Beleida formula to the same Nile water, but at a site differing as much as possible in depth and width, at the same discharge; even one of the branches below the Delta Barrage would afford a more valuable comparison than the Khannaq site.

The physical explanation suggested by Mr. Buckley was that the "blanket of slurry" referred to suppressed the formation of eddies by roughnesses on the bed, or their propagation into the mass of the flow, and that this in effect produced a smooth (and moving) bed for the main flow. Mr. Crump's suggestion (p. 272) was that silt in effect rendered the whole mass of water disinclined to eddy. Mr. Claxton's suggestion (p. 273) was that, while under given conditions a certain amount of initial eddying would be set up by the bed, the larger the amount of silt, the greater was the eddying energy used up in suspending it, and the less was left over for more thorough eddying that opposed flow. Mr. Gilbert (p. 274) observed that an increase of silt smoothed the bed. The point appeared to be more of theoretical than of practical interest, and might well be left for solution by data that failed to accord with the incorrect hypotheses; at present the only fact relevant to the point was that apparently silt in complete suspension mattered less than silt hardly even in saltation and on the margin of settlement; this fact, if correct, was best accounted for by Mr. Buckley's explanation, though Mr. Gilbert's observation was not contrary to it.

The Buckley-Wilson discharge-recorder not only (admittedly) ignored the effect of silt on velocity, but it also ignored any changes of silted bed or cross section; in fact, it assumed that discharge was a function only of gauge-level and slope. Mr. Buckley showed that the Nile in Egypt had an extraordinarily stable regime, sensitive to every change of regulation at Assuan; this stability was possibly repeated to some extent in the lower reaches of any similar large river, but must be accentuated by the lakes and swamps on the head reaches of the Nile. Similar stability was far from being found in the rivers and canals of the Punjab; there was evidence of slow changes in the rivers, possibly cyclic or possibly cumulative; even in Lower Sind such a change was being shown; the head reaches of distributaries even at some distance from the heads of their canal showed annual fluctuations, and changes sometimes occurred suddenly. To such conditions the recorder was not suited.

In Appendix A (p. 198), it was suggested that the warning of impending silting should be acted upon by increasing the slope of the canal. For their first 50 miles or so, Punjab perennial canals were climbing from the rivers in the trough of the valley, to the

Mr. Lindley. ridge from which they could command the land; in this length they ran at the flattest slope possible, and with no fall to spare for increasing the slope on occasion.

In Appendix D (p. 207), it was stated that "in the ideal case . . . all these lines would intersect in one common point." When more data, over a wider range of conditions, perhaps made a more general formula possible, it must not be overlooked that a family of straight lines might also either be parallel, or all tangential to an envelope; in fact, the exponents in the formula might also be functions of the silt.

The foregoing facts from Punjab experience could not compare as regarded precision with the measurements presented by Mr. Buckley; but as it was unlikely that similar precise data would be available from other sources, Mr. Lindley offered them for what they were worth. He was not without hope that the lead given by the investigation made in Egypt would be followed in the Punjab.

Mr. Olive. Mr. W. T. Olive considered that the conclusion that admixture of silt increased the velocity was reasonably to be expected, for fine wet clay was undoubtedly an unguent or lubricant. Anything tending to reduce friction (which meant accelerating velocity) and to increase mass or density increased the two components of momentum and would augment the flow of the river. The hydraulics of Messrs. Beardmore and Box would be difficult to supersede. He had observed a lamentable tendency, within recent years, to create new and elaborate formulas on the plea of exactitude, wherein were adopted coefficients having an extended range of choice according to individual judgment. Kutter's formula was a case in point, where *n*, the coefficient of roughness, now ranged, he believed, from 0·008 to 0·057; yet it was stated (p. 194) that it might vary very considerably in the same channel. Mr. Buckley gave 0·0275 (p. 200) and 0·057 (p. 195). The value, after all, was empirical. No absolute measure of roughness of a river-bed or canal was possible, whilst the variety was infinite. With floats $3\frac{1}{8}$ miles apart, as described in Appendix A, was the true surface-slope attainable even with the most accurate levelling? Might not the water-surface, in falls small as those, be affected by winds, the downstream float actually being raised by such heaping up? Would not velocities taken by current-meter—many taken in each cross section—be more reliable in obtaining the mean velocity than one observation of slope by absolute levelling? He questioned whether there was not something wrong with the records in the case of the Ibrahimia (p. 194), since the variation (error?) was allowed to be 21 per cent, Mr. H. CARTWRIGHT REID remarked that the rivers and canals in Southern India flowed almost entirely in an easterly direction. The hypothesis which Mr. Lacey enunciated—that the rotation of the earth caused any moving body on the earth's surface to be deflected to the right in the northern hemisphere—did not appear to be applicable to rivers so situated, since the tendency of any deflecting force would be to retard such a flow rather than to deflect it. In regard to rivers flowing in a northerly direction, he thought that the hypothesis was not correctly stated, since the deflecting force would in those cases act on the left bank of the rivers and not on the right. Many instances might be cited of the tendency of rivers to deflect to the left bank, besides those to which Mr. Lacey called attention. The first case he mentioned was the Palar river; but, as that had occurred in "geological" times, it was impossible to hazard any definite opinion as to the reason of the deflection of that river south of Madras. The diversion of the combined rivers Euphrates and Tigris, which in the days of "Sindbad the Sailor" reached the sea at Khor-abdullah, some miles south of the present mouth of those combined rivers (now known as the Shat-el-arab), was a striking example of the deflection of a river to the left bank. The site of the old mouth was now desolate; there were hundreds of square miles of mud and salt lagoons without a vestige of animal or vegetable life, whilst formerly, when watered by those great rivers, it was extremely fertile. Another example might be cited, namely, the river Ganges, which was stated to have broken across to the south-east, that was, to the left, in the sixteenth century.[1] The case of the Godavery river at Rajahmundry indicated that the river had an equal tendency to erode the left bank on which the town of Rajahmundry was built; and as an example of the deflecting force arising from the earth's rotation, the site where the regime of a river had been changed by the construction of a bridge was not a very satisfactory instance to select. In practically every river the tendency to cut its banks alternated almost equally between the left bank and the right, and in the tidal compartments of the Orissa rivers, the Mahanadi and the Brahmani (the lower part of which was known as the Dhamra river), there was no marked tendency to adhere to the right bank rather than to the left—indeed in the Dhamra river the principal stream now flowed almost entirely on the left-hand side of the Kalibhanj island. Mr. Reid suggested that before adopting a hypothesis as having any practical foundation, more demonstration than the examples suggested by Mr. Lacey was Mr. Reid.

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[1] Minutes of Proceedings Inst. C.E., vol. ccv, p. 20.

Mr. Reid. required. The tendency of the bed of the rivers to rise when their regime was altered by the construction of levees or flood-embankments required earnest consideration. Mr. Lacey had enunciated sound reasons, and his observations tended to show that some of the serious defects which were now being felt in many Indian rivers were due to a policy of attempting to deal with the river in sections rather than as a whole. The value of water in India for irrigation was so great that considerable changes (due to the taking of all the early rains into tanks or reservoirs) had been caused in the regime of the lower parts of the rivers, which had left the river in many cases a mere storm overflow ; on the other hand the irrigation-tanks caught large quantities of silt and were gradually filling up, thus causing great anxiety and expense to the landholders. The high value of land on the banks of the lower parts of the river had been the cause of the construction of flood-embankments, which, though in themselves commercially sound propositions, caused general interference with the river as a whole, the detrimental effects of which were not felt for many years afterwards. All the matters to which Mr. Lacey called attention indicated the disadvantages which arose from piecemeal treatment and the necessity for the conservancy of a river as a whole.

Mr. Rothery. Mr. S. L. Rothery observed that Mr. Buckley's contention that a heavier fluid or slurry existed against the periphery of a canal in which silty water was flowing, due to precipitation of solids resulting from the retarded velocity caused by friction with the canal-sides, at once raised the question whether such precipitation would be continuous along the whole wetted surface of the canal, and thus rapidly reduce its cross-sectional area, to the ultimate stoppage of flow conditions. Contrary to that hypothesis was the generally-accepted reasoning that to stir up and maintain in suspension any such high proportion of mud required a contact velocity such as did not exist near the bottom (*cf.* Dr. Herbert Chatley's silt investigations in China[1]). The lubricating effect on the periphery was apparent, however, with the flow of silty water, and was a factor that was partly responsible for higher velocities of flow. It appeared that the effect might be produced by the deposition of silt on the wetted surface giving a smoother surface than usual, due to the fineness of the silt particles that collected in the irregularities of the sides and bottom, or by the particles rolling along or among each other in the region of low velocity likely to be present near the periphery. Such deposition of a fine silty coating was evident in some places in the Imperial County canals in California, where seeped conditions of canal-banks and contiguous land happened immediately after a canal had been dredged; but after a short period of flow the seepage diminished and the banks became watertight again. Mr. Rothery.

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[1] Minutes of Proceedings Inst. C.E., vol. ccxii, p. 100.

Sir John Benton, in describing the Punjab Triple Canal System,[1] stated that Mr. Kennedy's non-silting channels had saved the Government of India vast sums of money. The annual cost of maintenance of Egyptian canals was £350,000, which, it was hoped, would be reduced by the campaign of silt-research organized in 1918. In Imperial Valley, California, turbidity in canal-flow was ever present, and the silt-content was two or three times that in Indian or Egyptian canals. The silt-burden was so heavy that the flow of the river and canals was quite silent, and bubbles and ripples were not to be seen on the surface. A practical study of canal-flows with the silt conditions in the Imperial Valley would undoubtedly be valuable and reduce dredging-maintenance in the future. *Fig. 17*, in his Paper (p. 179) and the determination of the increased value of the coefficient in Kennedy's formula, in order to render it most helpful in California, was due to Mr. W. A. Campbell, his assistant for 5 years, whose perception and knowledge of the conditions to be striven for on that project were invaluable. For the application of the proposed new Beleida formula to different rivers, the value of the constant 147 B_0 would apparently vary and would have to be determined for the minimum values of the silt-content, in order that the given coefficient and indices to the hydraulic radius and the slope could apply generally. The criticism and subsequent application of the new formula to general use, where turbid flow was controlled, should be watched with interest, as any such correction of velocities, which were as much as 50 per cent. in error when computed by hitherto-recognized formulas, must necessarily be of practical value to engineers.

Mr. Thorpe. Mr. W. H. THORPE found it difficult to accept Mr. Buckley's proposition as to the influence of silt upon discharge-quantities, yet the extraordinary closeness of results by the Beleida formula, which included that factor, to actual results, suggested that there must be some connection. Silt in suspension might, however, be only symptomatic of some other condition which was the real influence. This influence might be the changing of the character of the river-bed from time to time. In falling flood, and for months thereafter, the river-bottom being in some measure swept clear of loose deposits, hollows and obstructions were exposed, and greater resistance to flow might be expected, the water running clear.

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[1] Minutes of Proceedings Inst. C.E., vol. cci, p. 21.

Mr. Thorpe. On the other hand, when in flood the stream was heavily charged with silt; and though the power of the stream to transport it was evident, there would possibly be, at river-section in excess, a filling of hollows and general smoothing of the river-bed, favourable to easy flow. He suggested that clear water with high resistance and silted water with low resistance were simply indicative of a difference in the river-bed, which might be the true occasion of the facts observed.

Mr. Thrupp. Mr. EDGAR C. THRUPP agreed with Mr. Lacey's observations with regard to marginal erosion being assisted by ground water exuding from the soil during the fall of a river. That occurred on the Thompson river (Canada) at a place where the rise in normal seasons was about 14 feet. A much more serious cause of marginal erosion on large rivers was wave-action during high winds, which had been frequently noticed to cause more erosion in half an hour than the flow of the river alone would cause in a year. For 13 years he had counterbalanced such erosive action at the river frontage of his own property on the Thompson river by partially burying brushwood at or near the low-water line, and thus causing silt to deposit and flatten the slope of the bank. Mr. Lacey alluded to the theory of the rotation of the earth tending to make rivers flowing south erode their right or west banks. If such a theory were correct, there should be some evidence of it on the Fraser and North Thompson rivers in British Columbia. Along part of their courses they had twice cut down to their present beds, first in glacial times when they had left terraces of glacial drift high up the mountain-sides, and second after some huge obstruction had blocked their outlets for a time and filled up the valleys 300–800 feet deep with what was known as "white silt," the material for which was derived from the previous glacial-drift terraces and mounds. The second cutting down therefore took place through very soft material; and, if there was a law of westerly erosion, there was no excuse for those rivers disobeying it. Yet the North Thompson near Kamloops had almost wiped out the white silt on the east side, leaving only a trace of a bench at its highest level, and leaving benches several hundred yards wide on the other side of the valley. It now took a curious easterly turn before joining the South Thompson, whereas it might have cut through a low alluvial flat on the west side. The Fraser river between Lillooet and Lytton had also left wide benches on the west side, several hundred feet above the present river-bed, and it was not causing any conspicuous erosion at the foot of the slopes of the benches; in fact the remarkable stability of those slopes when washed to a gravel surface had surprised him often when passing down the Fraser Cañon, because the fall of the river

Mr. Thrupp.

was roughly 7 feet per mile, the ordinary flood-discharge being 300,000–400,000 cubic feet per second. Having regard to this evidence against the theory mentioned, he was inclined to conclude that the instances to the contrary in India were merely coincidences.

Mr. Buckley's Paper was a welcome addition to hydraulic experimental records, although Mr. Thrupp ventured to dissent from some of his comments and conclusions. Mr. Buckley criticized several general formulas because they showed discrepancies of 50 per cent. compared with his observations on the Nile, and he seemed to treat the matter as if it were a question of inaccuracy of coefficients of roughness and of exponents of the main factors. He omitted to mention that all general formulas were framed to refer to uniform channels : that was the only reasonable basis upon which they should be framed. Unfortunately, natural channels were neither uniform in section nor straight in alignment, and artificial channels subject to erosion and silting were not uniform and might not be straight for considerable distances. The effects of irregular beds and deviations of alignment were quite distinct from the effects of mere differences of roughness in the material forming the beds. No two rivers were exactly alike, but one characteristic perhaps was common to almost all, namely, that the obstructing effect of irregularities in the bed diminished as the depth of the water increased, or, in other words, the deeper the water, the nearer the approach to the ideal uniform channel. Hence it was unnecessarily misleading to work out coefficients to apply to low-water conditions. A nearer approach to uniformity would be found among high-water results. The low-water deviations caused by irregularities were very similar to those caused by weeds growing in uniform channels; and the Beleida formula without its silt factor corresponded in character with numerous records of weedy channels.

Mr Thrupp's conclusions with regard to the general laws of flow in rivers and canals were given in a diagram in the Plate accompanying his Paper "Flowing Water Problems."[1] In accordance with that diagram a uniform channel would discharge 12 per cent. more than at the high-water conditions at Khannaq (24th and 28th August, 1920), about 30 per cent. more at the 86 gauge-level, and 78 per cent. more at the 83 gauge-level. Again, a uniform channel would discharge 17 per cent. more than the high-water stage at Beleida, and 64 per cent. more than the low-water stage. Those high-water records were confirmation of the diagram. The Beleida formula and the Menufia Canal formula would not be satisfactory for general purposes, because

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[1] Minutes of Proceedings Inst. C.E., vol. clxxi, p. 346.

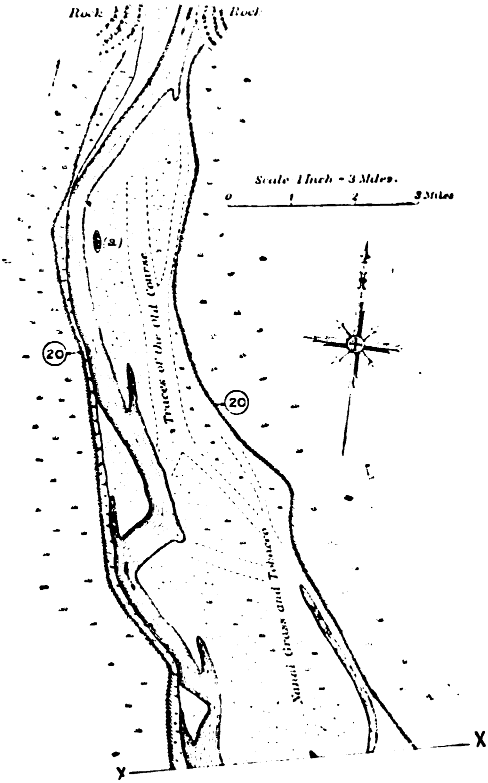
Mr. Thrupp. they simply represented local irregularities. In connection with the design of irrigation-canals the high-water discharge was of paramount importance. It was interesting to note that the ratios of mean velocity to maximum surface velocity observed in the Menufia canal agreed, apparently to within 2 per cent., with the figures given in the Paper previously referred to. The value of the record would be improved if Mr. Buckley would add the mean velocities at which each of the seven results were obtained, because the ratio varied considerably with the velocity as well as with the hydraulic radius; and a statement of the width of channel and wetted perimeter would enable the modifying influence of the shape of the channel to be worked out. A further elucidation was needed in the statement that silt-precipitation occurred in the Menufia canal with a velocity of less than 1·1 metre per second when 1,750 grams per cubic metre was being carried. What was the corresponding hydraulic radius? Mr. Buckley seemed to be in doubt as to whether the Beleida formula should contain the silt factor or not, for he stated that the increase in velocity was not due to the silt in suspension, and yet he called the factor a necessity. Mr. Thrupp agreed with the former statement; he believed that the increase was due solely to the deposited silt altering the degree of roughness, and that, if that influence could be actually determined, it would be found that the silt in suspension had precisely the opposite effect. The fact that the formula agreed with the data fairly well was no proof of the accuracy of the silt factor. A factor based on the number of days before or after the 1st September could also be made to fit the flood period. The discharge-recorder was ingenious, but, strictly speaking, the use of the logarithmic scale for representing the slope was not correct in the zone of conditions within which the instrument was being used. The exponent of the slope was a variable in that zone, as represented in Mr. Thrupp's general diagram, and as partially recognized in the Kutter formula. If the slope varied considerably, the assumed small percentage of defect or excess might be wrong. In that critical region the flow of water was not stable, but was influenced by meandering eddies; and it was the degree of turbulence caused by those eddies that determined the silt-carrying capacity. The bottom velocity was not the determining factor, as was often assumed. That was a big question, and could not be dealt with in a short discussion. It was not improbable that the actual velocities at a given stage of a river differed by a small percentage on rising and on falling conditions, owing to the meandering eddies working up in the one case and calming down in the other. Those eddies would occur in a perfectly smooth uniform channel, but were aggravated or accentuated by irregularities in the bed. Perhaps some time in

Mr. Thrupp. the future it might become necessary to study in detail the hydraulic conditions and silting problems towards the upper end of the Assuan reservoir, and then some new data might be obtained with regard to the critical velocity at which the change took place from pure stream-line flow to the meandering-eddy zone, and vice versa.

Mr. Lacey. Mr. Lacey, in reply, remarked that the objective in training rivers such as those described in his Paper should be to preserve the live channel of the river in easy forward-moving sinuous curves in the sand bed of the river; and in order to do so, and to maintain the river in such a regime, it was necessary to study the forces acting on the water flowing down the river, and the action of the water itself on the river-bed and margins. One of these forces was that due to the earth's rotation, and its action on the water flowing down a river was fully explained by Mr. Alford. This force might be small in low latitudes, but it had to be reckoned with.

Fig. 9.

(*Continued at* XX *in Fig. 10.*)

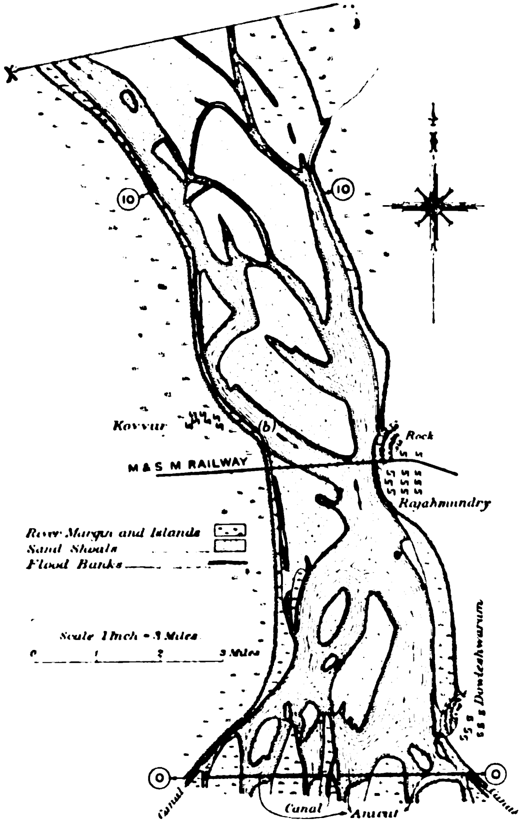


The survey (*Figs. 9* and *10*) of the Godavery river from the Gorge

Mr. Lacey. to the anicut at Dowlishwaram, taken after the flood season in the cold weather, 1919–20, showed that the live stream had become jammed against the right margin, and the left margin was accreting. The original flood-banks were constructed in the sixties or seventies, but their alignment had been frequently altered. There was also a local legend that the rock island, marked (*a*) in *Fig. 9*, containing the Patteswaram Temple, once formed the right margin of the river.

Fig. 10.

(*Continued at* XX *in Fig.* 9.


X
(10)
(10)
Kovvur
(b)
Rock
M & S M RAILWAY
Rajahmundry
River Margin and Islands
Sand Shoals
Flood Banks
Scale 1 Inch = 3 Miles
0 1 2 3 Miles
Dowleshwarum
Canal
Canal
Anicut
Canal

At Kovvur the line stream crossed over to the left bank, where there was always a deep pool against the rocky margin at Rajahmundry. Two causes might be given for that sudden change in the direction of the line stream. One was that at Kovvur there was a projection in the river-margin of a stratum of hard black clay marked (*b*) in *Fig. 10*, which was not so easily eroded as the more recent alluvium, and which possibly acted as a spur, deflecting the live stream across the river. The other was that during floods the velocity of the current in the deep channel on the Rajahmundry margin caused a retrogression of its bed and captured the live stream on the right margin. The railway bridge at Rajahmundry was built to the full width of the river and could have little influence on its regime. The anicut at Dowlishwaram apparently had had a stabilizing effect on the river. Mr. Lacey.

With reference to Mr. Reid's statement regarding the Ganges river, Mr. Lacey would draw his attention to some particulars given by Sir Bradford Leslie.[1]

The Paper dealt chiefly with that portion of a river which was above tidal influence. The tidal reaches and mouths of rivers were largely influenced by the direction and the force of the tides and the prevailing winds. It was possible that a study of the forces at work in the Thompson and the Fraser rivers might account for the conditions obtaining there. Wave-action was, as stated by Mr. Thrupp, a serious cause of marginal erosion in large rivers.

Mr. Rothery. Mr. ROTHERY, in reply, stated that on its delta the Colorado river did not change its course by moving up an incline as suggested by Mr. Lacey. Meandering about some general direction for several years, the river-banks and contiguous ground had become raised by, in general, about 1 foot per annum, so that after a decade the river was flowing on a ridge about 10 feet above the surrounding country. It began to overspill its banks at about 30,000 to 50,000 cusecs discharge, with a few inches depth of overflow at its banks, increasing to depths of several feet in the dense vegetation as the distance from the river increased. When this elevated condition had come about, some effluent channel, that had become formed down the slopes of the ridge, invited the falling-off of the river to a new course on lower country, when there resulted a definite change in direction, as in 1909, when the direct course to the Gulf was the elevated one. Since that year the diverted westerly course had become elevated probably to 12 or 15 feet, and in 1921–22 advantage was taken of this condition, and a diversion was produced down this south slope, which had turned the river to the left, away from part of the levee system.

A detailed description of how this diversion had been effected had been published in the Proceedings of the American Society of Civil Engineers.[2] Evidence of the oscillations of the Colorado river between

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[1] Minutes of Proceedings Inst. C.E., vol. ccv, p. 75.

[2] See footnote to p. 169, *ante*.

Mr. Rothery. the Gulf and the Salton Basin in recent geological times was apparent in the existence of these ridge slopes, which showed in cross-sectional topography on the basin soils, and was apparent also in the stratification and different textures of the fine soils that comprised the great bed of the Colorado river alluvium within the ancient beach-line of the Gulf. This beach-line was clearly defined, conspicuous from a distance, and extended north of the Salton Sea. The geology and soils of this basin, and two hypotheses relevant to the cut-off of the north end of the Gulf, by the river, had been given full consideration in a treatise[1] by D. T. MacDougal and collaborators, to which Mr. Rothery referred Mr. Lacey for a more complete answer to his question.

Mr. Buckley's discussion on the salient features of the Paper was instructive in the comparisons made with the two great rivers of Mesopotamia and with the Nile. The Colorado river was still in the process of forming its delta, and the physical records of the present era of man's interference on that delta would, in the far distant future, be buried deep under beds of alluvium yet to be transported to the delta, before physical geography there had become stabilized. Storage-dams would postpone this distant day; but even such large dams as that proposed for Boulder Cañon would become silted full in time; it was said the efficiency of that dam would be seriously impaired as a storage for flood-protection after a period of 160 to 200 years. Embankment of the overflowing river on both sides, and improving its regime by dredging and suitable training were expedients that would be resorted to in the present and in the early future, in order to develop the agricultural potentialities of this fertile delta. Rockwood gate was operated with flashboards, and care was taken to admit only inflow from the highest level of the water in the river; at peak demands, and when there were large volumes in the river, all of the seventy-four openings were used; with low river stages sand-bar conditions in front of the gate required the operation of the openings in a series of groups, and the shallowest possible overpour for the whole length of the structure could not always be obtained.

Mr. Buckley. Mr. BUCKLEY, in reply, referred those who had criticized the adoption in the Beleida formula of the quantity of silt in suspension to his reply to Dr. Unwin's remarks in the Discussion, which dealt with this question. Mr. Butcher would find therein a possible explanation of the case of the Rayah Tewfiqi. Mr. Buckley, however, would point out that the discharge diagram given by **Mr. Buckley** Mr. Butcher (*Fig. 4*) was of but slight assistance without the water slopes corresponding with the plotted discharges. Mr. Kanthack also would find therein a reply to the last sentence of his interesting contribution. Mr. Kanthack had never found that an excess of water was being delivered to the canal-system during the silt season, but Mr. Buckley would call his attention to that part of his reply to Sir Thomas Ward (p. 244) which referred to Indian and Egyptian canals, and would add that the difference between the coefficients meant that, as other conditions appeared the same, Mr. Kanthack's canals, if supplied with water from the Nile during the silt season, might carry about 40 per cent. less than they actually did. His reference to the experiments of Swiss engineers was of interest, especially the fact that the diminution of velocity, attributed to increased viscosity due to heavy charging of silt, appeared to have been observed mainly in the surface velocities. That fact was significant, as it would appear strange that the effect attributed to the load should be greatest in that part of the stream where the load must have been least. He fully concurred with the suggestion of the existence of sand-waves, and referred Mr. Kanthack to his reply to Professor Dixon, laying emphasis on his theory that, where the conditions were such as to permit of a considerable load being carried, the existance of that load, within limits, would tend to neutralize the eddies thrown off by the sand-waves, or, to put it another way, the eddies thrown off by those waves might be largely absorbed in supporting the load, leaving the rest of the channel in a less-agitated condition than it might otherwise have been.

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[1] "The Salton Sea," etc. (Published by the Carnegie Institution.) Washington, 1914.

The following passage from Mr. G. K. Gilbert's Paper, alluded to in the Discussion, referring to vertical velocity-curves, appeared significant to the Author, although Mr. Gilbert's conclusions seemed to be different :—

> "The plotted points for velocities near the channel bed are irregular when the observations were made above a bed of loose debris, and little use has been made of them in drawing the curves . . . . Not only was it impossible to observe with accuracy the relation of the instrument to the normal position of the bed, but the velocity observed was higher than that normally associated with the depth at which the instrument was placed" (p. 247).

Mr. Gilbert also, in describing experiments carried out with clear water flowing over " beds composed of fixed grains of debris or debris pavements," reported the following :—

> "It was thought that their mean velocities would be materially higher than those observed with streams otherwise similar but bearing

Mr. Buckley. loads. When the comparison was made, however, it was found that in eleven out of sixteen experiments the observed velocities were higher with loaded than with unloaded streams; and the average of the sixteen results was of the same tenor, ascribing a slight excess of velocity to the loaded streams" (p. 229).

This amounted to a substantial corroboration of the silt effect.

The fact—pointed out by Mr. Thrupp in his Paper on "Flowing-Water Problems"[1]—that Révy found that the bottom velocity of the La Plata increased faster than the surface velocity, might be accounted for by the silt influence. As a correct appreciation of all conditions could only be arrived at by weighing all available evidence, Mr. Buckley would place the evidence quoted in his reply to Mr. Beresford against the conclusions arrived at some time ago by the French and Swiss engineers referred to by Mr. Kanthack.

Mr. Christen[2] was of much the same opinion as himself, namely, that when conditions arose which caused silt to be removed from the bottom, the latter became rougher and opposed additional resistance to flow.

He would refer Mr. Thorpe to his reply to Professor Dixon and to the Indus figures quoted in his reply to Mr. Lacey.

He had dealt in his reply to Dr. Unwin's remarks with the question of the use of general formulas. A formula like Kutter's, which had been adjusted so as to agree with gaugings of the Mississipi, could hardly be said to be framed to refer only to uniform channels. His principal object had been to devise some means of determining the discharge of the Nile by a much less costly method than was possible by current-meter measurements, and it appeared that such an end could be achieved by the Beleida formula. The value of the hydraulic radius was a pertinent factor in the case where it was stated that silt-precipitation occurred in the Menufia canal with a velocity of less than 1·1 metre per second, when 1,750 grams per cubic metre was being carried; and it was for that reason that those data were coupled, in the Paper, with a gauge-reading of 15·40. The value of the hydraulic radius was 4·34 metres. The use of values of *R* observed at a single section of the river, coupled with the slope over a long reach, had been criticized by Messrs. Butcher and Hurst, who stated that the method would lead inevitably to erroneous results; but, as the former had produced an exponential formula which he claimed accurately represented the observed data, and which relied entirely on the values he criticized, his condemnation did not appear to be quite justified. Similarly, Mr. Hurst produced three formulas for the Nile which also relied on the criticized values of R and S. Mr. Hurst's Beleida curve, however, when compared with Fig. 11, Plate 3, showed that during the rising stage, when silt was brought down by the river, his formula gave persistently low values for the velocities, while during the falling stage, when the channel was scouring, the values given by the formula were persistently high; that demonstrated its inferiority to the Beleida formula, and its similarity to Mr. Buckley's original approximation referred to in the upper paragraph on p. 195. As regarded the Malakal conditions, water-samples taken of the Bahr-el-Gebel in March, 1923, and analysed at the Wellcome laboratories, Khartoum, showed that, whereas the silt-load at Juba, 100 miles from Lake Albert, was only 140 grams per cubic metre of water, it was 170 grams at Bor and 260 grams near Lake No. In other words, the Bahr-el-Gebel, which at that time of year had no tributaries of its own, and was the principal tributary of the White Nile at Malakal, was scouring. The silt-conditions at Malakal, therefore, were not stable, and Mr. Hurst's assumption of the absence of the silt effect might easily lead him to erroneous conclusions. Mr. Hurst, in introducing his formulas, laid emphasis on the fact that different constants were necessary for different stations on the river; but, as one of the principal features of the Beleida formula was its applicability without change of constants, Mr. Hurst's formulas could not be placed in the same category with it. Mr. Hurst attached importance to his statement that "the mean slope over the 3 kilometres at Khannaq ranged from 44 to 75 per cent. of the slope from Assuan." It seemed possible, however, that that statement might be misleading. *Fig. 11* (p. 294), compiled from Mr. Hurst's Table, showed that the relation between the local and the Assuan slopes followed a very definite curve, the maximum error of any of the points being less than 9 per cent.; so that Mr. Hurst's reference to differences of 75 per cent. seemed to lack significance. As explained in Mr. Buckley's reply to Dr. Unwin, the local slopes criticized by Messrs. Butcher and Hurst were used for computation in 1919; they yielded results analogous to those based on the Assuan slopes, which were published in August, 1920. **Mr. Buckley.**

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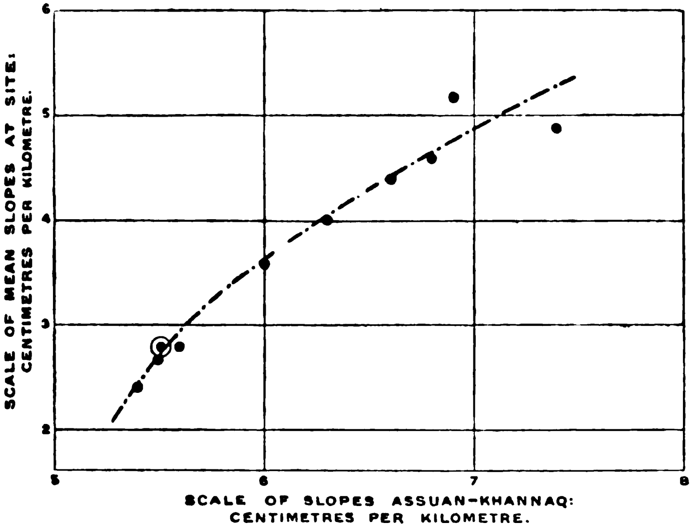
[1] Minutes of Proceedings Inst. C.E., vol. clxxi, p. 346.

[2] "Handbuch der Ingenieurwissenschaften, III Teil. Practische Hidraulic," Bubendei.

The figures for the 20th August, on which Mr. Butcher based his opinion that the error of the recorder during the critical silting period was almost certainly instrumental, were in no way conclusive, as inspection of Fig. 1, Plate 3, showed that the discharge on the 20th August was being rapidly reduced, and that the conditions of flow were unstable. In this connection reference

Mr. Buckley. should be made to his reply to Dr. Unwin. Since the recorder worked on the step-by-step principle, it necessarily followed that changes, when they occurred, did so in jumps. The jumps indicated alterations in water-level and canal-slope which, under those particular conditions, would be accompanied by silt-deposits quite compatible with sudden changes of velocity due to alterations of the eddying agitation at the bottom of the canal. As explained in the Paper, the 17-per-cent. excess of discharge in August was not determined only by means of the recorder, but was definitely established by the official irrigation discharges, the importance of the recorder lying in the fact that, without its intervention, this phenomenon would have eluded detection, as it was not the practice to record the canal slopes or to refer the official discharges to them.

Fig. 11.



The water-surface slopes of the canal which Mr. Hurst criticized were based on the readings of two electric gauges working on the "Telechron" principle, while the recorder was an entirely separate instrument worked by means of relays from the Telechron lines. The Telechron gauges were never found to be in error, and he was confident that the slopes which Mr. Hurst criticized were those which actually obtained. There were several probable explanations to account for the discrepancies to which Mr. Hurst attached importance, the most probable being that the canal gauges, which, owing to the distances which separated them, had necessarily to be read by

Mr. Buckley.

different watchmen, were not read simultaneously, either with each other or with the Telechron gauges. Moreover, it would be seen that columns 1 and 2 of Mr. Hurst's Table compared almost as unfavourably with each other as did columns 3 and 5, the differences being 9 per cent. in defect and 4 per cent. in excess, as compared with 9 per cent. in defect and 9 per cent. in excess, if the single instance of 14 per cent. on the 11th–20th September, which could be accounted for by fluctuating levels, was neglected. The period chiefly relied on in the Paper for the silt effect was from the 18th to the 27th August, not from the 20th August to the 15th September; Mr. Hurst's criticism was not applicable to the former period. Mr. Hurst's other references to the recorder performance rather tended to complicate the issue, which concerned a definite instance of increased discharge under silted conditions, duly authenticated by the official irrigation discharges, after having been brought into prominence by the recorder which, during the rest of the time, might equally have been out of action without influencing the argument. He hesitated to accept Mr. Hurst's statement that according to the argument in the Paper, if there was silt in the water, with a falling velocity, for the same values of R and S, the velocity was higher than with a rising velocity. That conditions of rising or falling velocity might influence the result had never been suggested. Mr. Buckley's argument, on the contrary, was that, when the Nile was rising, which period corresponded with one of rising velocity, for the same values of R and S the velocity might be greater than when the river was falling, the result being influenced by the silt. He referred Mr. Hurst to his reply to the Discussion which, among other matter, contained reference to the comment of the Indus River Commission on a similar phenomenon, mentioned by Mr. Lacey.

Mr. Olive referred to the possibility of the Ibrahimia records being in error; were that a solitary instance, the suggestion naturally would carry weight. According to the late Mr. P. à M. Parker[1] reliable observers had found variations as high as 35 per cent., which were attributed to silt influence.

Mr. Lindley's contribution would be welcomed by students of hydraulics. It called for no other remark from Mr. Buckley than an expression of appreciative acknowledgment.

The occasion seemed opportune for him to add the following example recently brought to light by the inspection of the irrigation

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[1] "Control of Water as applied to Irrigation, Power, and Town Water-Supply Purposes." London, 1913.

Mr. Buckley.

1 Date: 1922.	2 Silt in the Nile at Cairo.	3 Level Downstream Head.	4 Slope.	5 Mean Velocity.	6 *R*.	7 Kutter's *n*.	8 Discharge.
	Grams per Cubic Metre.	Metres.	Centimetres per Kilometre.	Metres per Second.			Cubic Metres per Second.
July 1	12	15·54	5·38	0·556	2·38	0·0245	50·5
„ 9	14	..	..	..	..	..	..
„ 22	48	15·55	3·53	0·524	2·38	0·0222	47·5
„ 29	14	..	..	..	..	..	..
August 1	..	15·83	5·84	0·677	2·6	0·0229	68·3
„ 5	96	..	..	..	..	..	..
„ 8	..	15·83	4·70	0·641	2·6	0·0218	66·0
„ 12	889	..	..	..	..	..	..
„ 15	..	15·75	4·06	0·680	2·6	0·0189	66·0
„ 19	2,046	15·75	3·30	0·682	2·53	0·0165	66·0
September 2	1,952	..	..	..	..	..	..
„ 9	1,884	15·75	3·67	0·644	2·54	0·0187	62·5
„ 16	1,524	..	..	..	..	..	..
„ 23	1,348	..	..	..	..	..	..
„ 30	1,320	..	..	..	..	..	..
October 7	1,162	15·65	3·31	0·588	2·38	0·0188	53·3
„ 14	954	..	..	..	..	..	..
„ 21	742	..	..	..	..	..	..
„ 28	704	..	..	..	..	..	..
November 4	664	..	..	..	..	..	..
„ 11	464	..	..	..	..	..	..
„ 18	382	..	..	..	..	..	..
„ 25	344	15·70	5·46	0·55	2·44	0·0261	51·5
December 2	244	15·50	3·46	0·497	2·32	0·0230	42·9
„ 9	236	15·60	3·77	0·55	2·38	0·0215	49·9
„ 16	202	15·50	2·69	0·496	2·32	0·0205	42·9
„ 20	..	15·75	6·15	0·59	2·46	0·0260	55·5
„ 23	105	..	..	..	..	..	..
„ 30	145	..	..	..	..	..	..

The velocities and values of *R* in this Table are measured at a site 800 metres from the head of the canal.

gauge-books and discharge-records, which illustrated the variations of Kutter's *n* in the Ismailia Canal in 1922, with varying silt-loads in the water. **Mr. Buckley.**

The Ismailia Canal derived its water from the Nile at Cairo. At a distance of only 13 kilometres from the head was a main regulator, whose up-stream water-level was raised to the maximum permissible limit, as soon as the flood arrived in sufficient abundance, for the benefit of a certain small branch canal whose total discharge amounted to 4 per cent. of the discharge entering the canal. For any given discharge, therefore, the slope was governed by the level held up-stream of the regulator at kilometre 13, and the coefficient of roughness of the canal. The diminution of this factor with the arrival of the silt-laden water was evident from column 7. It was opportune that the period of rapid increase in silt-load from the 8th to the 19th August coincided with a period of constant discharge in the canal—66 cubic metres per second—for under these conditions the flattening out of the slope from 4·70 to 3·30 centimetres per kilometre was a direct indication of the reduced roughness of the canal-bed, as it was known that the canal was not scoured out to a section which would correspond to the reduced slope.

The foregoing returns had been compiled with the assistance of Mr. H. F. Ayres, B.Sc., Assoc. M. Inst. C.E., and Mr. Buckley was indebted to Dr. C. Todd, O.B.E., Director of the Public Health Laboratories, for permission to use the silt-determinations.

As perusal of the foregoing might convey the impression that Mr. Gilbert's own conclusions were fundamentally different from Mr. Buckley's, it was necessary to emphasize that not infrequently Mr. Gilbert made reference to a possible physical law which might have escaped his analysis, and he suggested a hypothesis which involved horizontal vortices tending to account for variations in the level of the point of maximum velocity in a vertical. This appeared to be a specific instance of Dr. Unwin's conditions of "eddying agitation," which Mr. Buckley had suggested would be modified by the silt slurry. While the developments of the experiments which Mr. Gilbert had proposed for testing the validity of his hypothesis would be generally instructive, Mr. Buckley suggested that the silt influence in this respect, as defined in his Paper, might be greater than had hitherto been supposed.

In conclusion, he desired to record his thanks to those who had contributed to the Correspondence. To have the benefit of the opinions of practical engineers with experience of the behaviour of rivers and canals was most advantageous, and it was interesting to note that some of them had had experience of the

Mr. Buckley. silt effect. The concordance between Mr. Duponchel's conclusions, Mr. Gilbert's experiments, and his own, appeared sufficiently remarkable.

13 March, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The Discussion on the Papers by Messrs. Lacey, Rothery, and Buckley was continued and concluded.

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20 March, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

On the recommendation of the Council, the members present elected by acclamation as an

Honorary Member

Sir RICHARD TETLEY GLAZEBROOK, K.C.B., M.A., D.Sc., F.R.S.

(*Paper No. 4361.*)

"The Improvement of the Water-Supply to the City of Hobart, Tasmania; and the Manufacture of Reinforced-Concrete Pipes by the Centrifugal Process."

By the late JOHN CLARK ROSS, M. Inst. C.E.

WATER-SUPPLY.

THE city of Hobart is situated in the south-eastern part of Tasmania, at the base of Mount Wellington and on the estuary of the Derwent river, about 15 miles from the sea. It is probably the finest deep-water port in the Commonwealth, being well sheltered and easy of access, and having extensive and deep anchorage. It is fairly centrally situated with respect to Australia and New Zealand, and a depth of 50 feet at low water exists practically alongside the streets. It is hoped that eventually it may become a distributing-centre for overseas trade.

During 1911–12 the City Council decided that it was necessary to improve the water-supply. The population then served numbered between 30,000 and 35,000, and the supply was obtained from streams and springs rising on Mount Wellington and on the adjacent table-lands. The daily consumption of water during the dry part of the year reached about 2 million gallons. Mount Wellington rises to a height of 4,170 feet above sea-level, and the highest point is about 5 miles from the centre of the city; the slopes run southward and westward with pronounced valleys and peaks. The streams arising from these high lands have a heavy discharge during the wet season, but decrease during the summer. The springs which are tapped however, maintain, as a rule, a fairly uniform flow, though they are quite insufficient in themselves to supply the city.

Conservation is therefore necessary, and at the dates mentioned two reservoirs existed, one of 40 million and another of 60 million gallons capacity. The water was led to these by cast-iron pipes, ranging from 10 to 16 inches in diameter, and by masonry channels and aqueducts 16 to 18 inches square in section.

The gradients of these pipes and channels were very steep in some places and flat in others. This scheme was initiated during the early development of Hobart, and was well designed and effective under the circumstances then existing. As the population increased, however, the supply-channels were extended from time to time to connect with other sources. As the original scheme had been taken to such a high level, the extensions had to be laid to gradients so flat that the size of pipe which the city could afford was insufficient to carry anything like the supplies that were available. To remedy matters it was decided to build a large new conservation-reservoir to feed those in existence, and to lay a new supply-main thereto. The existing reservoirs, which were close to the city, one at about 450 R.L. and the other at about 500 R.L., acted both as distributing-reservoirs to the higher parts of the town and as supply-reservoirs to two smaller distributing-reservoirs in the city proper. Mr. R. S. Milles, who was City Engineer previous to 1913, was instructed to prepare a scheme in accordance with the decision arrived at; and, after exhaustive surveys, a site was chosen at Ridgeway (about 4 miles from the centre of the city) immediately above the existing reservoirs, at an elevation of 900 R.L. This site was the only one that was even fairly satisfactory. A wall retaining a maximum depth of water of 90 feet conserved only 134 million gallons. Prospecting shafts were sunk and tunnels were driven to ascertain the character of the foundations. They disclosed what were believed to be reliable foundations in solid diabase. The design of the dam was then proceeded with by Mr. Milles in conjunction with the late Mr. L. A. B. Wade, M. Inst. C.E., of Sydney, who acted as consulting engineer. Various designs were considered, and the plan eventually adopted was the one under which the work has been carried out.

This design consisted of a composite dam of concrete with two straight gravity wing walls and an arched centre wall. Owing to the configuration of the country, the dam is on the far side of a saddle from the city, the outlet to the existing reservoirs being driven through the intervening rise. The work was let by contract. As the foundation excavations proceeded, the character of the rock disclosed was found to be not nearly so good as had been anticipated. The ground had been badly shaken, and, on exposure, large boulders got loose and came down. The worst rock was in the lowest part of the valley, and here the excavations were carried down more than 100 feet. Further, in this deep part of the excavations a fault was disclosed about 8 feet wide, running at right angles to the line of the wall, which was filled with compacted crushed rock. This fault was excavated to a depth of 85 feet, making a total depth of 185 feet below the natural surface of the ground. At this depth the sides of the fault were still dropping almost vertically, but, as the material excavated was so densely compacted, it was decided to commence the foundations, and the fault was concreted up to the bottom of the main excavation.

At this stage (1913) Mr. Milles resigned, and the Author, being appointed in his place, took over control as a complete stranger. It was soon found that a pessimistic feeling existed in the minds of the public and of the Council with regard to the works; but, after analysing the situation, the Author felt that the difficulties had been exaggerated. The progress of the work, however, had been so slow that to expedite matters the Council determined the contract and carried out the construction departmentally.

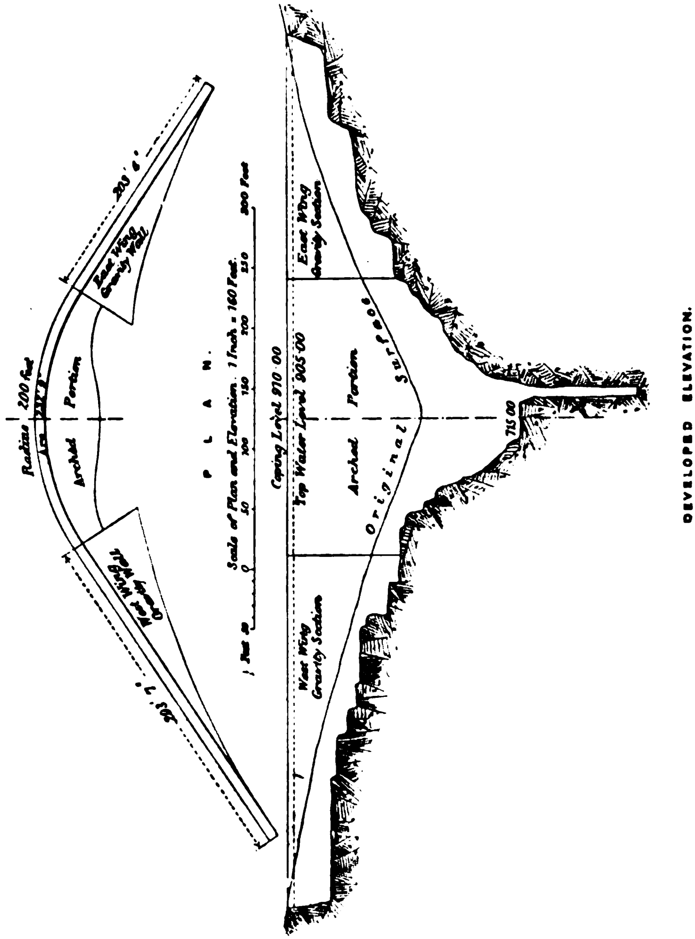
The whole question was then reconsidered, and the original design of the work was confirmed by the Author with one alteration, namely, the raising of the wall by 10 feet, thus increasing the capacity of the reservoir from 134 million gallons to 207 million gallons. Legal proceedings delayed the recommencement of the work for nearly a year. The design of the dam, as altered and completed, is shown in *Figs. 1, 2,* and *3*. The leading dimensions are as follow:—

	Ft.	Ins.
Length of west wing (gravity wall)	293	7
,, arch	232	9
,, east wing (gravity wall)	203	4
Total length of dam	729	8
Radius of arch	200	0
Maximum depth of arch, top to foundation	201	0
,, ,, ,, to bottom of fault	285	0
Maximum depth, east wing	105	0
,, ,, west ,,	103	0
Top width of wall below coping	6	0

Water area when full	19 acres.
Maximum depth of water	95 feet.
Capacity, full	207,371,500 gallons.
Height above sea-level (T.W.L.)	905 feet.

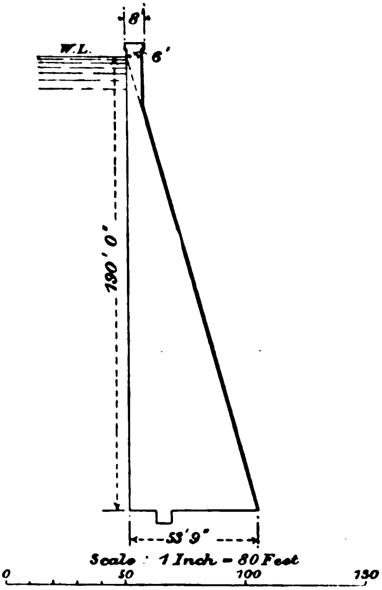
The gravity wings and arch are built independently of each other. A 6-inch by 6-inch bitumen joint with sheet-lead faces is carried from ground-level to the top of the wall, forming a watertight tongue, recessed partly into the wing and partly into the arch. The curved part of the dam is reinforced by two laminæ of 40-lb. rails arranged with a 6-foot mesh for about two-thirds of the height. A reinforcement consisting of four rails is also run from end to end of the wall below the coping. The whole of the wall is built in concrete—6 to 1 for face work and 8 to 1 for heart work. All concrete was mixed mechanically in batch-mixers, being elevated and poured by shoots for the most part. Some was placed by skips handled by a 5-ton "flying fox" of 800 feet span, which was installed to remove excavations. The stone used for concrete was the diabase obtained mostly from the foundations, and the sand was got by crushing sandstone found near the work. A mixture of 3 parts of

Figs. 1.


PLAN.
Radius 200 feet
Arc
Arched Portion
East Wing Gravity Wall
West Wing Gravity Wall
203' 6"
203' 1"
Scale of Plan and Elevation. 1 Inch = 160 Feet.
Feet 50
0
50
100
150
200
250
300 Feet
Coping Level 910·00
Top Water Level 905·00
East Wing Gravity Section
Arched Portion
Original Surface
West Wing Gravity Section
715·00
DEVELOPED ELEVATION.

sand to 1 part of Union cement (Sydney) had a tensile strength of 275 lbs. per square inch at 7 days. The cement used was practically all of Australian make and had a tensile strength of 800–1,000 lbs. per square inch at 7 days. Plums were used where convenient, but it was found cheaper to omit them unless they were of such size that they could be easily handled by the men.

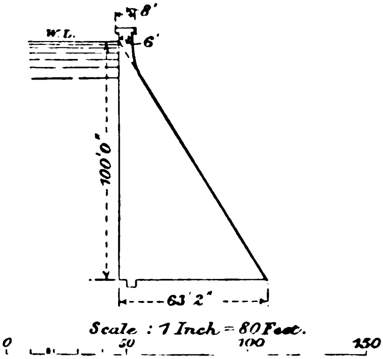
Fig. 2.



Typical Profile of Arched Portion

The water is carried from Ridgeway to the lower reservoirs in a concrete tunnel through the saddle, the water passing the first section from the bottom of the reservoir to the concrete valve-tower in the tunnel itself, and from the valve-tower to the open in reinforced-concrete pipes—a total distance of about 1,000 feet. No overflow is provided, as the supply is maintained by pipes and can be controlled. An 18-inch Venturi meter is placed on the inlet-pipe to register the inflow, and it is proposed to place meters on the four outlet discharge-mains from the lower reservoirs to register the daily consumption.

Fig. 3.



Typical Profile of Gravity Wall.

Considerable delay in carrying out the work—apart from legal proceedings—was caused by bad weather and by difficulty in obtaining cement and plant, owing to the war. The total excavation for the dam amounted to 45,000 cubic yards, and the concrete placed was 47,425 cubic yards. The average cost of crushed stone at the mixer was 6*s*. 5*d*. per cubic yard, and of sand, 6*s*. 6*d*. per cubic yard. The cost of cement varied from 11*s*. to 20*s*. per cask, with an average of about 15*s*. per cask, f.o.b. Hobart. The average cost of concrete placed was 29*s*. per cubic yard.

The whole cost of the reservoir, including the valve-tower and outlet-tunnel, etc., but exclusive of supply- and discharge-pipes, law costs, road-diversion, and catchment-drains, amounted to £116,500.

INTAKE AND PIPE-LINES OF REINFORCED CONCRETE.

The principal source of supply is the North-West Bay river, the head waters of which are impounded by a small weir at an elevation of 2,050 R.L., about 9 miles from Ridgeway reservoir. The bed of this stream is steep and rugged, and, when floods occur, boulders of all sizes up to 3–4 tons are brought down. The weir is a low structure of concrete about 4 feet by 4 feet in section; it is built in solid rock at an angle to the course of the stream. At the up-stream end, sheltered by a rock bluff, an intake-chamber is built, from which a supply-pipe is taken.

Originally it was intended to carry either a steel or a cast-iron pipe-line from this weir to Ridgeway, along the slopes of the mountain, following as nearly as possible the existing pipe-line, and picking up various streams and springs on the way. From Ridgeway also a discharge-line was required to supply a distributing-basin from which other lines branched to the lower reservoirs, city mains, etc. These various lines ranged in diameter from 10 inches to 18 inches, about one-half of the 9-mile main from the North-West Bay river to Ridgeway being 15 inches in diameter and the other half 18 inches in diameter. As the original section for these lines was prepared for cast-iron or steel pipes, high pressures at various points and flat gradients at others were involved. On tenders being called for these lines in either cast-iron or steel, the prices quoted were so great —owing to war conditions—that, under the Author's advice, it was resolved to inquire into the possibilities of centrifugally-made reinforced-concrete pipes. After careful inquiry, inspection, and tests, the Author felt justified in recommending their use in place of iron or steel for the particular work required, stipulating that a new and suitable line should be designed. The recommendation being adopted, a new route was surveyed and levelled, and a pipe-line was designed to carry a maximum of 10 million gallons per diem. This line was contoured for the greater part of the way to the hydraulic gradient.

The length was divided into four sections, with break-pressure tanks between them. These tanks were of concrete, 10 feet in diameter and 10 feet deep, with cross walls and grids to break the flow. The delivery-pipe from each tank was either taken at a slightly steeper gradient than the supply-pipe, or made of larger diameter; no valve was placed at the delivery ends to the tanks, but valves were placed near the top ends of the supply-pipes to the lower tanks. Between these valves and the upper tanks a scour-valve of a capacity equal to the flow-pipe was placed, leading to adjacent creeks or drains. By opening these scours the valve on the main could be closed without putting any great extra pressure on the line. This was done in order to avoid the high pressures that would otherwise obtain, the total head exceeding 1,200 feet when Ridgeway reservoir was nearly empty. By this means no head could come on this line greater than that due to the difference of level between the scour-pipe and the hydraulic gradient. This head amounted in places to 25–50 lbs. per square inch.

In order to save length, a gully 800 feet deep was crossed, but this section was laid with steel pipes ranging from $\frac{7}{16}$ to $\frac{9}{16}$ inch in thickness. Throughout, air- and scour-valves were placed where necessary.

On the 15-inch line between Ridgeway and the distributing-basin to the lower reservoirs a valve was placed at the lower end, which entailed a pressure of over 110 lbs. per square inch on these pipes.

The main from the intake to Ridgeway was designed for, and tested to, 75 lbs. per square inch, and the line between Ridgeway and the distributing-basin was similarly treated for a test of 130 lbs. per square inch, the other lines being tested to 100 lbs. per square inch.

All the pipes were made in lengths of 6 feet, joined by a loose concrete collar. The ends of the pipes have a tapered recess in which is placed a special plastic compound, like bitumen in appearance and character, to seal the joint under pressure. When pipes have been laid, they are jacked up tightly together, the collar is slipped over the joint, and the annular space between pipe and collar is packed solid with a 2-to-1 sand and cement mixture only slightly moistened. This joint sets rapidly and does not shrink appreciably. The pipes and collars are of 2-to-1 mixture, and the walls range in thickness from $1\frac{1}{8}$ inch for 10-inch pipes to $1\frac{3}{8}$ inch for 18-inch pipes. The reinforcement is proportioned to the test pressures required and is usually of 10- or 12-gauge steel wire. The concrete in these pipes is very dense, and the inside and outside surfaces are beautifully smooth. No difficulty was experienced in laying either in hot or in cold weather, and, although the pipes were exposed for months to sun and frost, very little trouble was caused by the variation of temperature, only a few joints having to be remade owing to that cause. A number of pipes had to be rejointed owing to careless workmanship, but the whole number of remade joints did not amount to 2 per cent. Since the work was completed, little or no maintenance has been required. Practically the whole length of pipes laid is curved. Where very sharp bends occur, cast-iron specials are used; but, by using short bevel-ended pipes, fairly sharp curves of 30 feet radius for 15-inch pipes can be negotiated, while full-length pipes were laid repeatedly on a 100-foot curve.

The lowest price quoted in 1916 for 15-inch steel pipe was 14s. per foot, f.o.b. Hobart, and for 18-inch cast-iron pipe, £11 10s. per ton f.o.b. Hobart. The cost of the main intake-line would have amounted to £28,824, exclusive of cartage, laying, and jointing, while the cost of the same line in reinforced-concrete amounted to £13,624, and this price included cartage, laying, and jointing. Both amounts are exclusive of earthworks.

It is believed that the use of concrete pipes will not only avoid corrosion troubles, but that they will improve with age.

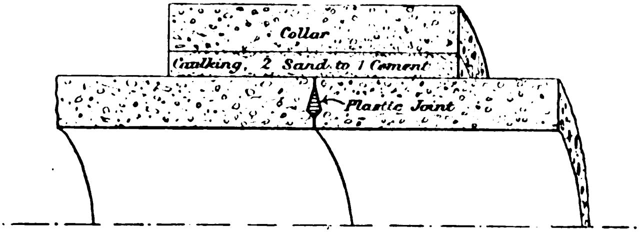
THE MANUFACTURE, USE, AND CHARACTERISTICS OF REINFORCED-CONCRETE PIPES MADE BY THE CENTRIFUGAL PROCESS.

The Author does not consider that concrete pipes can displace iron or steel for all purposes, but in many cases they can be used with advantage and economy. Apart from the question of cost, which seems to be clearly in their favour, there are other reasons why their use deserves serious consideration. They are strong, they have a certain amount of elasticity, they may be overstrained without serious loss of efficiency, they are less liable to corrosion than is steel or iron, and, when centrifugally made, they are light in weight. They are easily and cheaply laid, and they can be manufactured on the site if necessary.

These pipes are made in standard lengths of 6 feet, from 4 inches to 15 inches in diameter, and in lengths of 8 feet above 18 inches in diameter, or in any shorter length. The range in size up to the present has been from 4 inches to 8 feet in diameter, though neither size is the limit; smaller sizes can be made, but not economically, while offers have been made to manufacture up to 15 feet in diameter. The collars for these pipes are made in lengths of 6 to 12 inches, depending on the diameter of the pipes; they are reinforced similarly to the pipes and have the same thickness of wall, the diameter being increased to allow sufficient clearance for caulking. Where small service-connections are required to be taken off these pipes, screwed steel sockets are spun in the wall of the pipe; and where larger branches are required, either saddle-pieces, bored and screwed, are used, or ordinary cast-iron tees, crosses, etc.

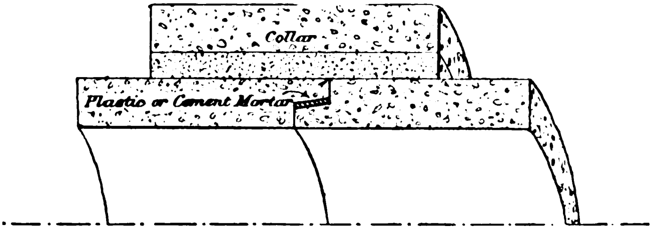
The ends for jointing are of various types. *Fig. 4* shows a pipe end with a taper recess—the most usual type. This recess is filled with a plastic compound which is somewhat similar in nature to bitumen, and which, when internal pressure is applied, seals the joint. *Fig. 5* shows a telescopic joint used for light pressures or for drainage, either with or without a collar; it is also used for borehole-lining without collars. *Fig. 6* shows a joint for large pipes; *Fig. 7* an improved type of joint which allows freer movement for expansion and contraction. In the standard joint the outer surface of the pipe is left smooth and the inside of the collar is roughened so that the caulking adheres firmly to the collar, while a slight movement of the pipe may occur without leakage. The space between the collar and the pipe is packed tightly with a mixture of 2 parts of sand to 1 part of cement, moistened to about the consistency of brown sugar. This mixture is caulked in solidly against an angle-ring support fixed temporarily on the barrel of the pipe, next to the collar.

Fig. 4.



Approximate Scale: One Quarter Full Size

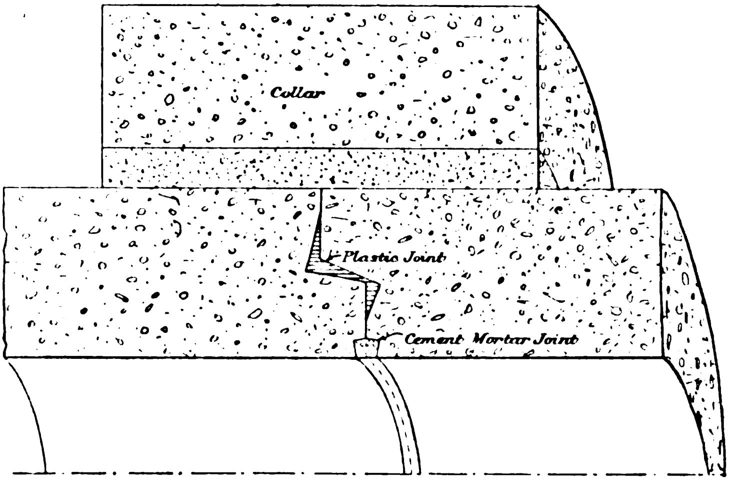
Fig. 5.



Approximate Scale: One-Quarter Full Size

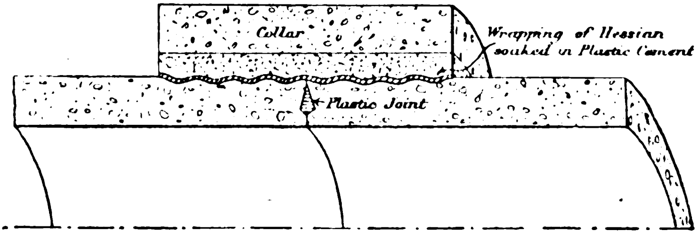
Manufacture.—In the case of the particular type of pipe with which the Author is concerned, both pressure and non-pressure pipes are made in an exactly similar manner, and both are reinforced. The pressure pipes are composed of 2 parts of clean sand to 1 part of cement; the non-pressure pipes of 2 parts of small screened stone, 2 parts of sand, and 1 part of cement. The concrete is mixed rather wet in batch-mixers before being placed in the moulding-machine, which rapidly extracts most of the water. The usual reinforcement is mild steel wire of gauges ranging from No. 12 to No. 6. In calculating the amount of reinforcement required for pressure pipes it is wise to limit the tension in the wire used to 12,000 lbs. per square inch, except in

Fig. 6.


Collar
Plastic Joint
Cement Mortar Joint

Approximate Scale: One-Quarter Full Size.

Fig. 7.


Collar
Wrapping of Hessian soaked in Plastic Cement
Plastic Joint

Approximate Scale: One-Quarter Full Size pipes less than 12 inches in diameter, where 15,000 lbs. per square inch may be safely allowed.

In large pipes—3 feet or more in diameter—the 12,000-lb. limit should be reduced. These limits are purely arbitrary, but tests prove that the smaller the pipe, the higher the permissible limit. This is no doubt due to the circumference of the reinforcement in the smaller pipe suffering less total extension than in the larger type, thus straining the concrete shell to a less degree. The value of using steel having a high elastic limit is doubtful, as the concrete shell of the pipe will crack before the elastic limit of lower grades of steel is reached.

The wire for the reinforcement, both of pipes and of collars, is wound on expanding steel drums, and is fed through a travelling eye geared to lay automatically on any pitch. The necessary diagonal, brace, and longitudinal wires are placed on by hand, then further layers of spiral reinforcement, etc., are laid on until the required weight of steel is made up. The drum is then caused to collapse, and the finished cage is removed endwise. The reinforcement is wound ¼ to ½ inch less in diameter than the outside diameter of the pipe. The cages are placed in light galvanized-iron tubes made in two or more sections keyed together. These form the moulds for the pipes, and are coated freely inside with residual oil before being used. On the ends of these tubes two cast-iron flanged runner wheels are placed and bolted together with outside longitudinal tie-rods. The projection of the inner flange of these end rings determines the thickness of the wall of the pipe.

The spinning-machine on which these moulds are placed consists of a bed frame on which are mounted a series of three to seven parallel axles with cast-iron disks at each end. The spacing of these axles is variable, to suit the different sizes of pipes. The end rings of the moulds rest on and between the revolving disks, which act as friction driving-rollers, the first set of axle and disks only being directly driven. The motion is communicated by friction to the rest of the series. The machine is run at a low speed to start with, ranging from 150 revolutions per minute for 4-inch pipes to 48 for 6-foot pipes. The concrete is shovelled into the revolving moulds, and is distributed uniformly with a little assistance. The machine is then speeded up to 700 revolutions per minute for 4-inch pipes and to 120 revolutions per minute for 6-foot pipes. After running for 8–12 minutes, the machine is stopped, and the water which accumulates in the centre is swabbed out. The machine is again started, and an iron bar is placed through the pipe; this moves backwards and forwards while bearing on the concrete; finally, it rests on the surface of the inner flange of the end rings. This gives a uniform thickness of wall and a smooth internal finish. At this stage —12 to 15 minutes from the commencement of spinning—the concrete is so firm that it is difficult to make an impression on its surface with the thumb. The actual time required for spinning ranges from 8 to 20 minutes or more, even though the conditions are apparently exactly similar. The reason for this variation has baffled all inquiry. Analysis of the cement and water, testing of the sand, variation of climatic conditions, etc., have failed to account for it. The pipes themselves do not appear to vary in strength or character with different setting-times. Further investigations of this phenomenon are being made.

After spinning is completed, the mould and pipe are passed into a steam curing-chamber, where they are subjected to a saturated atmosphere of 130° to 150° F. for about 8 hours. After removal from this chamber the mould is stripped off, and the pipes are stacked. The pipes for pressure purposes are either covered over with wet sacks and sprayed continuously or placed in water-tanks for some days. Various other methods of curing are adopted, depending on circumstances, but the best results are obtained by keeping the pipes damp and cold. In favourable circumstances pipes 24 hours old ring like a bell on being struck, and will withstand a very considerable test, both internal and external.

As pipes are made, a certain proportion of each type is tested up to or beyond the limits for which they were designed. It is very seldom that a pipe fails within these limits; as a rule the tests show a margin for internal pressure of 100 per cent. in the smaller sizes to 25 per cent. in the larger.

Table I, taken from works records, gives a few results of tests. Except where bursts are noted, the figures do not represent the limits of pressure which the pipes will withstand; but they are sufficient to show their capabilities. A useful feature of these pipes is that, even if a pipe is strained by extreme pressure to the point of fracture, at a lower pressure the pipe may be quite serviceable. For instance, a pipe that was designed for 50 lbs. per square inch split under a pressure of 250 lbs. per square inch, but held at 150 lbs. per square inch. This, of course, is quite explainable; and it is a very useful feature, inasmuch as an important pipe-line, whilst rendered defective under unusual circumstances, may still remain quite useful.

Experimental pipes have been made to withstand much higher pressures than are usually required, by spinning a steel or sheet-iron liner in the centre of the wall of the pipe. Reinforcement is placed

TABLE I.—INTERNAL HYDRAULIC-PRESSURE TESTS.

Mixture: 2 sand, 1 cement.

Diameter.	Number of Pipes.	Thickness of Wall.	Age.	Wire in Pipe.	Designed for	Tested to	Remarks.
Inches.		Inch.	Days.	Lbs.	Lbs. per Sq. In.	Lbs. per Sq. In.	
4	Single	1	20	5	60	225	Burst.
4	,,	1	24	11	150	160	Sound.
4	Two-jointed	1	9	20	280	225	,,
4	,,	1	40	20	280	380	,,
6	Single	1	25	8	50	100	,,
6	,,	1	35	15	100	225	Burst.
8	Two-jointed	1	35	62	225	250	Sound.
8	Single	1	70	62	225	300	,,
9	,,	1	50	22	100	175	,,
10	Two-jointed	1⅛	35	18	50	100	,,
10	Single	1⅛	28	37	100	125	,,
12	,,	1 3/16	23	26	50	65	,,
12	,,	1 3/16	40	80	125	200	,,
18	,,	1⅜	30	74	50	55	,,
18	,,	1⅜	45	150	100	110	,,
18	,,	1⅜	80	183	120	190	,,

TABLE II.—EXTERNAL TESTS.

Diameter.	Thickness of Wall.	Wire per Pipe.	Age.	Load per Foot Run.	Remarks.
Inches.	Inches.	Lbs.	Days.	Lbs.	
4	¾	Nil	35	830	Crushed.
4	¾	1½	17	2,130	Sound.
4	¾	1½	40	2,150	,,
8	1	6	22	1,600	,,
8	1	12	42	2,200 2,900	Slight crack invert. ,, ,, soffit.
10	1⅛	7½	36	1,200	Slight crack invert.
15	1¼	11	..	1,100	,, ,, ,,
18	1⅜	21	..	500 1,200	,, ,, ,, Crack, invert, sides, soffit.
21	1½	37	117	890 2,300	Crack invert. Cracked all round but holding.
30	1¾	45	87	600 2,000	Crack invert. Cracked all round but holding.
[1] 36	2	45	85	3,514 7,514	Slight crack invert. Cracked all round but holding.

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[1] This test was made with the pipe in a trench, covered with 18 inches of filling on top of the pipe and with the load distributed over the pipe.

in the internal and external concrete shells. This liner has a loose hooked longitudinal joint with sufficient play to allow it to expand and to bed itself into the concrete while spinning.

Table II (p. 311) gives the results of crushing tests of pipes made for sewerage and culvert purposes. These pipes are made of 2 parts of stone, 2 parts of sand, and 1 of cement, and for general purposes are lightly reinforced to a standard. For pipes to sustain any special external stress the reinforcement is made ovoid and is increased to suit the circumstances, the wall of the pipe being thickened as required. All the tests noted in Table II, except where otherwise stated, were made on lengths of 3 feet, the pipe-barrel resting on a bed of sand without any side support. The load was applied on a 2-inch steel bar resting lengthwise on top of the pipe to be tested. The weight of wire given is the weight put in a standard length of pipe, i.e., for pipes up to 15 inches in diameter, 6 feet, and above 15 inches, 8 feet.

Tests made for absorption in accordance with the conditions laid down in the British Standard specification for salt-glazed earthenware pipes gave the following results:—

ABSORPTION TESTS.

Thickness of Pipe-Wall. Inch.	Permitted Increase in Weight. Per Cent.	Actual Increase in Weight. Per Cent.
1	6	4·33
1½	8	5·12
1¾	10	5·59

The specific gravity of the concrete in pipes made by the centrifugal process has been found by repeated trials to be about 2·50.

This class of piping is suitable for many purposes—well-lining, borehole-casing, sinking cylinders, culverts, sewerage, and water reticulation, irrigation supplies, pumping and gravity mains, telephone conduits, telegraph and telephone pole-bases, and tanks for various purposes. For mine-ventilation on the Rand, pipes up to 23 inches in diameter and ½ inch thick, reinforced with wire netting are being used; they are taking the place of galvanized iron and canvas pipes, although their weight is rather against them.

Experiments are being carried out, with fairly satisfactory results, to ascertain their suitability for oil and gas lines. Trial lines also have been laid to see whether they can be used with advantage for the discharge of the liquid refuse from sugar-works (dunder), and of acid water from mines, where iron and steel pipes have been destroyed, in a very short time. It is too early to speak with any certainty as to results, but no deterioration has been observed with 6–8 months' service.

The light ventilation pipes have also been used as a 40-foot chimney-shaft for a boiler. This shaft is 23 inches in diameter, with a ⅝-inch wall, and has been in use for nearly 12 months without the least sign of a crack.

Although centrifugally-made concrete pipes may, generally speaking, be used with advantage and economy in most water-carriage systems and the like, there are certain conditions under which particular care must be exercised in order to obtain satisfactory results :—

(1) Where the foundations are bad, thorough supporting and packing are necessary, owing to the rigidity of the joints.

(2) When great variation of temperature occurs, either internally or externally, concrete-packed joints tend to draw and weep. In such cases metal expansion-joints may be used, or alternatively, the joint itself may be made with lead, carefully caulked. The lead may be in the form of wool, or it may be melted, run in, and caulked.

(3) When very high pressures are required in pipes of large diameter, a metal liner may be of advantage, although pipes 5 feet in diameter have been designed, and are being made in the ordinary way, for a pressure of 100 lbs. per square inch.

The Author's experience has shown that the ordinary joint with plastic sand and cement has proved excellent for most purposes, if carefully made ; but, where it is necessary to make a repair quickly, a run joint of bitumen, lead, or fusible cement may be advantageously used to obviate delay.

The Paper is accompanied by two sheets of tracings, from which the Figures in the text have been prepared.

(*Paper No. 4366.*)

"The Mazoe Irrigation-Dam."

By Mark Randall, Assoc. M. Inst. C.E.

Southern Rhodesia, a land possessing vast agricultural and ranching possibilities, includes no more promising area than this important and conveniently-situated valley (Figs. 1 and 2, Plate 5), and it is here that the chief efforts of the British South Africa Company have been directed in establishing irrigated agricultural estates. In the province of Mashonaland, of which Mazoe is a district, maize-production, dairy-farming, pig-raising, citrus-, tobacco-, and cotton-culture are chiefly practised, whilst in the southern half of Mashonaland and in Matabeleland—the remaining province of Southern Rhodesia—ranching and pastoral farming find more favour.

The Mazoe River Valley.

The Mazoe valley proper begins at the Poort, some 21 miles due north of Salisbury. The site of the dam in the Poort has an altitude of 4,010 feet. Immediately below the dam the valley opens out into level stretches of land of good depth and great fertility, formed by the soil washed down from the gold-bearing diorite kopjes dominating the valley. This continues for 50 miles from the Poort to the Shamva gold-mines. Occasionally the older granite formation, and with it granite soil of poorer quality, are met with. The area of the Mazoe valley watershed from the Poort to the region of the Shamva mines is about 900 square miles, constituting the most important maize-producing centre of the whole territory; besides providing about one-third of the total crop, it also gives heavier yields per acre than any other district. In 1919 the official figures for the production of maize in Southern Rhodesia were 995,000 bags for the territory and 335,000 bags for the district of Mazoe. For a distance of 60 miles from the Shamva (3,000 feet altitude) towards the Zambesi river (1,000 feet altitude) the geographical formation is very broken indeed, and it is doubtful whether any agricultural development on a large scale could be undertaken there economically. Adequate railway facilities are provided by the Mazoe branch of the Rhodesian railways, which runs practically through the middle of the area from Salisbury to the Shamva mines, thus affording ready means of reaching the world's markets via the Port of Beira. The Rand and the Conga markets are also served by efficient railway-systems, whilst other railway-construction schemes are in contemplation, which will bring Mazoe into closer touch with the ports on the west coast than it is at present.

The European holdings at present in the valley average about 3,000 acres each in extent, chiefly devoted to mealie-growing. This crop yields good profits to the farmer only in favourable years. The growing season is from November to February or March, which corresponds with the season of heavy rainfall. For the remaining 8 or 9 months of the year the lands lie fallow. Under irrigation of a permanent nature it is anticipated that two crops in 3 years can safely be counted upon. These need not necessarily be maize: in fact the cultivation of rotation crops would be much more desirable for obvious reasons. Thus, owing to the certainty of water-supply, a reduced area of land would, in all probability, yield the same profits, besides affording cultivation for a greater range of crops.

In 1912 a syndicate—Messrs. McIllwane, Simpson, and the British South Africa Company—constructed a weir on the Mazoe river at the site shown in Fig. 1, Plate 5, known as Watt's weir. The canal took out on the right bank and was designed to irrigate a few hundred acres of citrus-groves on the farms Smithfield and Brundret. At that time, probably owing to the paucity of available hydrographic data, it was confidently expected that the normal flow of the river would be sufficient. After two or three seasons of practical experience it was found that not more than 50 acres could be irrigated, especially in years of low rainfall. In 1914, the commitments of the syndicate being fairly heavy, it was decided to attempt the augmentation of the seasonal flow of the river during periods of heavy irrigation demand.

In that year the British South Africa Company, which became the sole owners of the concern, embarked on a comprehensive programme ot citrus development, which included the provision of a large storage-dam, canal-construction, and the laying-out of a further 1,500 acres of fertile land on these farms for citrus-groves under irrigation. The scope of the programme was still further increased by subsequent purchase of lands on the farms Bloomfield, Clifton, Virginia, and Sleamish, so that the present extent of the irrigable area belonging to the Company is in the neighbourhood of 4,000 acres. The conditions are most favourable for successful development, as the control and the administration are invested in one company, which owns the storage-dam, the distributing-system, and the land to be developed, especially as the last is in one continuous block. It is intended to cultivate citrus, tobacco, cotton, maize, and other highly-remunerative crops on these lands. On the left bank of the river is an area of 5,000 or 6,000 acres of similar land belonging to private owners, and it is proposed to bring this area into the scheme.

HYDROGRAPHIC DATA.

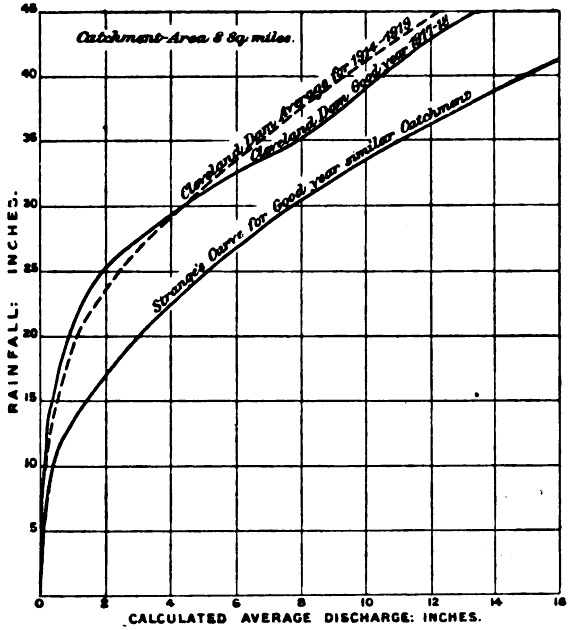
At the Poort the river has a catchment-area of 130 square miles, with altitudes above sea-level of 4,800 feet at the headwaters, and 4,040 feet at the Poort. The conformation consists of fairly open grass flats in red schist soil to within 6 miles of the dam. From here steep granite hills merge into the diorite kopjes forming the immediate flanks of the Poort. The whole catchment-area, with the exception of cultivated land, is covered in the wet season with a wealth of thick grass 10 feet or more high. The discharge, therefore, is considerably retarded, so that its incidence does not correspond with that of the rainfall, in spite of the fact that a fair proportion of the area is planted out to maize. As the crop reaches its maximum growth in the early part of the wet season, it helps to preserve this condition of deferred discharge.

An analysis of the annual rainfall (Appendix A) gives returns from stations in the catchment-areas of the Mazoe dam and of the Cleveland dam (Fig. 1). The municipality of Salisbury draws its main domestic supply from the latter. Suitable hydrographic measuring-appliances have been in constant use on the Cleveland dam area since 1912, and the data collected provided a valuable source of information upon which the hydrographic possibilities of the Mazoe catchment were based.

The average rainfall of the Mazoe dam catchment is 33·46 inches per annum; the maximum, 55·76 inches; and the minimum, 22·65 inches. The records extend over a period of 23 years (Appendix B). The returns from some of the stations are not complete, as the observations were made by farmers acting in a voluntary capacity; regularity is not expected under such circumstances. Nevertheless, all the figures quoted are taken from the Government records.

Appendixes C, D, and E show that, until the area has become saturated, the intensity of rainfall has only a minor influence on the discharge; and it appears that 18 or 20 inches of rainfall is the saturation-coefficient for the area. Whilst the seasonal fall is below that amount, any precipitation, however intense, will only give a discharge of 2 or 3 per cent. *Fig. 3* illustrates this point. The relation determined by Mr. W. L. Strange, M. Inst. C.E., relates to a similar catchment.[1]

Fig. 3.



In the case of Appendix C, which gives the discharges for 1917–18, the records were based on data supplied by farmers, etc., who, of course, used no exact measuring-appliances; and a big margin of error must therefore be allowed. During 1918–19 the Author directly observed the discharges tabulated in Appendix D. From

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[1] "Indian Storage Reservoirs with Earthen Dams." London, 1904.

July, 1919, the records (Appendix E) were based on actual measurements made by the Author of water-depth in, and discharge from, the reservoir. Due allowances must therefore be made for the important factor of saturation of the reservoir-bed, and in this respect it will be noticed that the actual discharge percentage is much lower for 1919–20 than for 1918–19. It is possible that, after September, 1919, when the dam outlet-gates were first closed, the whole reservoir-bed would absorb water to a depth of, say, 30 feet. On this assumption the amount absorbed would be 13,500 acre-feet (900 acres by 15 feet, allowing for 50 per cent. voids in the soil-formation), and the total discharge for 1919–20 would therefore be 39,320 acre-feet, or 16·20 per cent. of the precipitation. Although it is recognized that this assumption is open to doubt, the percentage arrived at is in accord with *Fig. 3*.

In calculating the inflow into the Mazoe dam, evaporation was allowed for at the rate of ⅓ inch per day from August to December, 1919, and ¼ inch per day from January to July, a rather higher rate than that experienced in the case of the Cleveland dam. The figures for evaporation in that dam for 1913–20 are shown in Appendix F. The average annual amount is 90·03 inches, the daily average being 0·24 inch. Daily records of evaporation are observed by means of an evaporating-pan. The average water-surface in the Cleveland dam is 100 acres, while its average depth is about 16 feet.

THE MAZOE DAM.

Site.—A contour survey of the Mazoe, Dassura, and the Tatagura rivers was carried out under the direction of the Author in 1914 and 1915. From the data obtained, five or six sites were selected for comparative consideration. The Mazoe Poort site (Fig. 2, Plate 5) was finally adopted after mature deliberation, in which several well-known Union and Rhodesian irrigation engineers took part. Trial-holes and shafts were sunk into the foundation and flank rock, and a fair indication of the general features was obtained. In view of the interesting means that were devised to render the rock impermeable and safe to withstand the stresses imposed by a solid structure of the type adopted, the foundations are described in detail. Mr. B. H. Maufe, the Rhodesian Government Geologist, reported, with regard to the foundations :—

> "The quartzite rock is a massive grey, finely crystalline rock, consisting essentially of quartz and carbonite (probably not pure calcite). It also contains disseminated crystal of sulphide minerals and very little chloride, and there are indications that the structure is not original. At

present I can only call the rock a quartz-carbonite rock—as in view of the large amount of carbonite it might mislead to call it a quartzite . . . . The ironstone is proved by the trial holes and excavations to rest on the quartz-carbonite rock. It is fissured by the numerous cracks which are mainly horizontal and open . . . . Over the third location of the foundations I consider the ironstone itself as sound, that is, between fissure and fissure. I understand that the François cementation process is to be applied to this rock. If any clay in the fissures can be washed out before the cement is injected a thorough cementation will afford an impervious mass of great strength."

The third location was adopted as the site for the foundations.

Design of Wall.—After consideration of several proposals, such as an earth embankment, a gravity dam, an arch-panel dam, etc., it was eventually decided to construct a solid dam, arched in plan (Figs. 4, Plate 5). The adoption of the Mazoe Poort site really depended upon whether the treatment of the permeable ironstone rock, comprising the foundations and flanks of any dam built across the gorge, could be successfully achieved. The alternative measure of excavating down to the felsite and rebuilding in concrete or masonry was considered prohibitive in cost. The spillways on the flanks (Figs. 5, 6, 7, 8, 9, and 10, Plate 5) were designed to take a maximum rate of discharge of 20,000 cubic feet per second with a corresponding rise in lake-level of 5 feet. It is considered that a further rise of 2 feet over the spillways and also over the main wall would not render the latter structure unsafe. The maximum known flood-discharge occurred in February, 1918, when the peak rate was about 10,000 cubic feet per second of 5 hours duration. An elevation on the up-stream face, and a sectional plan, of the sluice-chambers are given in Figs. 11, Plate 5. Fig. 12 is a section of the arched portion of the wall where its thickness is increased over the sluice-chambers. Fig. 13 is a plan of the sluice-operating platform.

Capacity of Reservoir.—The irrigation-scheme under consideration allowed the capacity of the dam to be based on average rainfall figures then available, as it was thought that the incidence of a dry year would represent merely a reduction in the crop for that particular year, corresponding with the shortage of water, and that there would be no permanent injury to the citrus trees. The data for 17 or 18 years in the Mazoe area and in the neighbouring areas indicated that the variation of rainfall from the mean was not very wide. Average annual rainfalls of 30 inches, with a 9 per cent. discharge yielding 18,700 acre-feet, were therefore adopted as probable mean values. The spillway-crests were placed at R.L. 4130, 90 feet above the lower outlet-culverts, or 98 feet above lowest foundation-level. Data, as disclosed by later observations (Appendixes A, C, D, and E) indicate that the assumed average annual rainfall was low, as was also the percentage discharge, and that, if the former amount were raised to 33·46 inches and the latter to 16·20 per cent., a better forecast of the possibilities of the catchment would be obtained.

The following Table gives particulars of the capacity of the dam, the lower outlet-level being 4040·00 :—

Contour above Sea-Level.	Depth of Water.	Surface Area.	Capacity.	
	Feet.	Acres.	Acre-Feet.	Million Gallons.
4,130	90	790	17,500	4,750
4,135	95	900	21,700	5,905
4,137	97	..	23,700	6,345
4,140	100	1,100	26,700	7,255

Cementation.—Figs. 4, Plate 5, show the positions and reference-numbers of the holes as finally treated, and Appendix G gives the particulars of the treatment, depth, and amount of cement used in each hole.

Treatment of rock around mine-shafts, in order to render it impermeable with a view to the elimination of underground water, is a process that has been extensively and successfully carried out for a good many years in coal-mines in England by the François Cementation Syndicate. In the last 5 years some gold mines on the Rand have also been treated by the Syndicate. Mr. A. François visited the site in 1917 and reported favourably on the prospects of treating the ironstone rock effectively under his firm's process. The contract for the cementation of the foundation-area was eventually carried out by the Syndicate. It is believed that this is the first occasion on which the process has been applied to any dam, on the scale and in the manner here carried out.

After the removal of the vegetation and loose ground, the exposed ironstone surface was marked out and trimmed for the reception of the Sullivan " H " drill, or the Beauty " E " drill, both of which were steam-driven. All holes were diamond drilled, vertically, with crowns from 1¾ to 2¾ inches in diameter. Ordinary percussion rock-drills were used for a few feet, but they proved unsatisfactory. Drilling was stopped as soon as 2 or 3 feet of core from the underlying felsite was brought up at any hole. A short pipe was inserted 2 feet into the top of the hole and made firm with cement, its projecting upper end being screwed to receive the flexible connection from the pump.

Two special cementation-pumps were used, one for each flank. They had 14-inch diameter steam cylinders, 1⅝-inch rams, and a 12-inch stroke. Messrs. Joseph Evans and Sons supplied one (Type 654), and the other was made entirely in Johannesburg by Messrs. Wright and Boag. A mixture of 3 parts of cement to 100 parts of water was used for ordinary work. This was kept constantly stirred up in the mixing-barrel by means of horizontal blades rotated by hand, and it was fed directly to the cementation-pump. The connections between the pump and the holes to be treated were of ordinary 2-inch piping with about 10 feet of flexible hose at the drill-hole end. Only one hole at a time was treated from each pump during pumping-operations. Pumping was carried out continuously in 8-hour shifts. The number of strokes per minute varied from 30 to 80, and the pressures from zero to 1,750 lbs. per square inch. The pump-man is highly-experienced, and specially instructed in this kind of work; the rate of feeding of the grout, the pressures to be applied, and the successful filling of the rock crevices served by the hole, depend entirely on his judgment. The chilled ball pump-valves controlling the grout supply have to be constantly cleaned and regulated, while the pipe-connections, owing to frequent cement-incrustations, must be kept clear. Inattention to these points causes a rise in pressure, with a corresponding loss of energy. Three or four applications were given to each hole at intervals of some weeks. Each application lasted a day or two. Special fine Portland cement (not more than 4 per cent. residue on 180 by 180 mesh) was used throughout. The initial setting-time was 100 minutes, and the final 400 minutes, as guaranteed by the Premier Portland Cement Company of Bulawayo, who supplied the special cement as well as the ordinary cement used for the structure.

During pumping-operations interconnection between the holes was very often set up, showing that the grout was finding its way through the cracks and crevices of the ironstone. Water under little or no pressure was first applied to the hole for some hours; then slightly higher pressures were applied gradually, until it was evident that the crevices were cleared of clay and dirt, as far as could be judged. It was interesting to note the different places where water issued from the exposed formation during this operation. Sometimes it would be seen spouting out of a newly-drilled hole 50 feet higher up the gorge slope; at other times it would issue from places much lower down. The points of issue were all carefully observed, and, after thorough flushing, were tightly caulked by means of cotton waste and wooden splinters. When no more water could be seen issuing from any place, after, say, 50–100 lbs. per square inch pressure was applied, the hole was ready for its first grout-application. Clay was stamped down into the hole for two-thirds of its depth, the upper third being left open. Cementation was then applied, and the upper layer of the ironstone received treatment. The pressure was not high at this stage, as it was necessary to guard against uplift of the formation. After a week or so the cement in the hole was drilled through by hand percussion, the middle third of the clay core was extracted, and cementation of the middle layer then took place. As the treated upper layer acted as a capping, a much higher pressure than was usually given for the first treatment could be applied with safety. The same processes were repeated for the third or fourth layers, with increasing pressures, until the entire hole had been treated. The C and F series of holes were treated in this manner, but, in addition, 2-inch pipes were brought up from them to the crest-level of the dam as the structure was built. This permitted further cementation to be carried out at any future time, should it be found necessary, without having to drill into the structure. A log was kept scrupulously throughout the progress of the work.

Cementation was first commenced on the 20th June, 1918, and continued uninterruptedly until the 20th November, when the influenza epidemic affected the labour-supply. It was resumed again in May, 1919, and was completed on the 15th September of that year. The "carbons" used were Brazilian borts of good quality, the wastage amounting to about 40 per cent. of the original. A total of 3,166 feet of drilling was done, and 3,868 bags of cement, or 360 tons, were used up in the process. Dr. Voskule, of the Central Mining and Investment Corporation, Johannesburg, represented the François Cementation Syndicate, and had charge of the field operations. The costs of the cementation are given in Appendix H.

Construction.—On account of the scarcity of good stone and sand near the site, a 2-foot gauge light railway was constructed out to a granite kopje some 3 miles to the south-east of the site, where very good construction sand in large quantities was found. The granite was blasted and quarried out of the slopes, broken up into "nigger head" size, and conveyed with the sand to the site of the dam by a steam locomotive and eight trucks. Forty tons of material was hauled per train-load, the number of return trips averaging five per diem. The stone was fed to a No. 323 Hadfield crusher, broken, screened, and shot into the hoppers feeding the tipping trucks which conveyed the graded stone to the concrete-mixer.

The sand was cleaned in a mechanical sand-washer of local design, which was most efficient. An Archimedean screw, made up of quadrants or blades of ⅜ inch by 4-inch iron, and having a propeller sweep of 2 feet 6 inches in diameter with a 40-degree pitch, was mounted on a 2½-inch square shaft. The latter sloped into the washing-box and was driven by bevel-gearing at its upper end; the lower end of the shafting worked in thrust blocks. Unwashed sand was fed in at the lower end of the box; the clean-water feed-pipe was at the upper end, and, as the water travelled against the pitch of the screw, it washed out all the clay and dirt, but left the coarser particles composed of sand. This was conveyed to the upper end, where it fell into hoppers ready for transmission to the mixer. A particularly good class of clean sharp sand was thus obtained at small cost. All the plant was driven by steam.

In the main wall, concrete blocks, cast near the site, were employed in lieu of shuttering. These were of convenient sizes to be handled by the masons. Two or three courses of each face of the wall were constructed and, after setting, the concrete hearting was then placed (Figs. 14, Plate 5).

The concrete for blocks was hand-mixed of 4-to-1 quality, while that for the hearting was machine-mixed of 6-to-1 quality. All stone and sand were carefully graded and measured with the water before entering the Ransome 10-cubic-foot mixer, which was situated at the right flank of the wall at a convenient level. The tipping trucks were hand-hauled on light rails, and the track was lifted as the work progressed. As much as 100 cubic yards a day was mixed and placed, but the average maintained was 70–80 cubic yards for a full mixing day.

The spillways were constructed in shuttering, but beyond this the above processes were applied. All concrete faces exposed to water-action were protected by the 4-to-1 grade concrete to a depth of 1 foot 6 inches (Fig. 15, Plate 5). No plums were used in the main structure and spillways.

The spur walls did not form part of the original design for dealing with overflow waters, but they were placed as shown in Figs. 4, Plate 5, at the direction of the consulting engineers, on the completion of the main contract. Owing to the steep sidelong slopes of the floor of the spillways it was thought that some division of the discharge, in order to break up its intensity near the spillway-toes, was necessary.

Concrete was first placed on the 1st November, 1918, and continued almost uninterruptedly to the completion on the 29th March, 1920. The influenza epidemic of 1918 delayed progress, but did not stop it entirely. An abstract of cost for the completed structure, which includes cementation, is given in Appendix J. During the flood period of January–February, 1920, several heavy discharges took place. The heaviest—that of the 14th February—passed an estimated quantity of 7,000 cubic feet per second at its peak, which lasted for 5 hours. The wall had reached a level of 4,056, and was entirely submerged; but very little damage was done, only a few of the facing-blocks laid the day before being washed away or unseated.

Leakage.—The sluice-gates were first closed on the 30th September, 1918, when water was stored in the dam. At about the end of October, with a top water-level in the dam of 4,078, a slight leakage, amounting to 0·015 cubic foot per second, appeared in the exposed felsite of the river-bed opposite the centre of the wall and about 40 feet from the down-stream toe. As the water-level rose, other leaks developed, all well down-stream from the wall and on both flanks of the river. Altogether about twelve main points of leakage were observed, and their positions and discharges were carefully recorded from time to time in correlation with the rise of water in the dam. Salt tests were also carried out in order to discover the points of entry of water in the bed of the basin. At certain observed places rough salt (in ½-lb. bags) was dropped into the water, and all leaks were tested for reaction with silver nitrate. Time and strength of reaction are the chief factors upon which considerations of the leak streams through the rock are based. Appendix K gives the leak-discharge from October, 1919, to July, 1920, and it will be seen that a great improvement took place. In April, when the total leak-discharge was at its maximum, it was thought that a reapplication of the cementation process would be required. The heavy leaks—Nos. 11 and 12—both on the left flank and issuing from the ironstone at above 4,090—gave maximum discharges of ½ to 1½ cubic foot per second each in April, and these were responsible for nearly three-quarters of the total leakage. It is considered that the leak-passages are well away from the foundations of the dam in the flank of the hill. Leakage has decreased considerably at these two points, while the remaining leaks show some small diminution. The total discharge of the leaks represents a small percentage of loss in the storage-efficiency of the dam; further, all water passing through the outlet-pipes, together with the leak-discharge, is picked up at Watt's weir, ¼ mile down-stream from the dam. That the leaks are decreasing, notwithstanding the increased head of water in the dam, portends that only a very small total amount of leakage need be expected in the future, and that further cementation will not be necessary.

Canals.—The Author was also employed on the design and construction of the main canal through Smithfield, Brundret, and Bloomfield, from Watt's weir. By the end of August, 1919, when he left the works, 9½ miles of canal had been excavated and completed. The canal has a bed-width of 4 feet 6 inches, 1-to-1 side slopes, and a fall of 1 in 1,000, and it discharges 60 cubic feet per second at the headworks. The soil is sufficiently tough to resist erosion, even though a velocity of 2·2 feet per second is sometimes attained.

Messrs. Sir Douglas Fox and Partners and Sir Charles Metcalfe, Bart., were the Consulting Engineers.

Messrs. Pauling and Co., Ltd., were the contractors for the construction of the dam, Mr. R. Pizzighelli being in residence as Agent.

The Author acknowledges with thanks the services of Mr. C. L. Robertson, by whose courtesy most of the hydrographic and rainfall data were obtained.

The Paper is accompanied by three photographs, two tracings, one print, and one map, from some of which Plate 5 and the Figure in the text have been prepared.

[APPENDIXES.

APPENDIXES.

APPENDIX A.

ANNUAL RAINFALL FOR STATIONS (ANALYSED).

Name and Number of Station.	Year.	Total Rainfall.	Rainfall during		No. of Rain Days During Year.	Greatest Rainfall in 24 Hours.
			Nov.-Mar. Inclusive.	Dec.-Feb. Inclusive.		
		Inches.	Inches.	Inches.		Inches.
Mazoe No. 1	1913-14	23·42	20·29	18·59	49	2·52
	1914-15	36·48	35·28	31·00	65	4·30
	1915-16	27·94	24·44	18·40	54	1·54
	1916-17	27·00	24·61	16·35	68	2·21
	1917-18	41·36	40·33	31·33	94	2·09
	1918-19	38·53	36·13	31·49	65	3·02
	1919-20	37·91	33·33	24·51	82	2·00
	Average	33·29				
Salisbury No. 4	1913-14	28·28	24·61	22·99	70	2·75
	1914-15	33·42	31·64	26·09	96	3·42
	1915-16	23·49	19·15	12·73	69	2·80
	1916-17	26·08	20·68	11·09	83	2·19
	1817-18	41·39	40·04	31·45	113	3·17
	1918-19	36·52	33·91	27·43	81	1·79
	1919-20	33·88	31·44	21·52	..	..
	Average	31·86				
Cleveland Dam No. 8	1912-13	31·24	28·70	23·42	91	2·60
	1913-14	24·79	22·08	19·23	72	2·61
	1914-15	33·87	32·56	27·09	84	2·15
	1915-16	18·06	13·32	8·71	67	1·47
	1916-17	20·02	17·00	11·45	76	1·20
	1917-18	45·16	44·47	32·03	121	2·85
	1918-19	33·16	31·03	23·86	92	2·77
	1919-20	39·41	36·45	23·15	90	2·50
	Average	30·70				
Volynia No. 9	1916-17	33·93	28·24	18·72	71	2·20
	1917-18	47·18	46·17	36·27	110	3·10
	1918-19	39·39	37·93	30·76	66	3·40
	Average	40·69				
Borrowdale No. 10	1916-17	25·28	21·72	15·42	76	1·60
	1917-18	55·76	54·57	37·52	99	4·62
	1918-19	32·55	31·54	26·33	70	2·42
	1919-20	37·55	35·07	24·44	61	4·45
	Average	37·78				

APPENDIX B.

TABLE OF COMPARATIVE RAINFALLS OF SALISBURY TOWN AND MAZOE CATCHMENT.

Year.	SALISBURY. Station No 4. Observed Rainfall.	MAZOE CATCHMENT. Observed Rainfall.	MAZOE CATCHMENT. Calculated Rainfall after adding constant of 2·03 Inches.[1]	Difference: Fall for Year minus Average Fall.	Remarks.[2]
	Inches.	Inches.	Inches.	Inches.	Year.
1897–8	27·58	no records available	29·61	−3·85	Bad.
1898–9	38·45	” ”	40·48	+7·02	Very good
1899–1900	37·17	” ”	39·20	+5·74	Good.
1900–1	41·78	” ”	43·81	+10·35	Very good.
1901–2	36·78	” ”	38·81	+5·35	Good.
1902–3	20·62	” ”	22·65	−10·81	Very bad.
1903–4	30·60	” ”	32·63	−0·83	Normal.
1904–5	30·22	” ”	32·25	−1·21	”
1905–6	27·82	” ”	29·85	−3·61	Bad.
1906–7	38·28	” ”	40·31	+6·85	Good.
1907–8	28·31	” ”	30·34	−3·12	Bad.
1908–9	38·10	” ”	40·13	+6·57	Good.
1909–10	31·25	” ”	33·28	−0·18	Normal.
1910–11	33·88	42·11		+8·65	Very good.
1911–12	21·98	25·45		−8·01	Very bad.
1912–13	32·74	34·45		+0·99	Normal.
1913–14	28·28	23·42		−10·04	Very bad.
1914–15	33·42	36·48		+3·02	Good.
1915–16	23·49	27·94		−5·52	Bad.
1916–17	26·08	27·00		−6·46	Bad.
1917–18	41·39	41·36		+7·90	Very good.
1918–19	36·52	38·53		+5·07	Good.
1919–20	37·55	37·91		+4·45	Good.
Average for 1910–1920	31·43	33·46			

[1] Average annual difference of 2·03 inches in favour of Mazoe Catchment.

[2] SUMMARY.

VERY GOOD YEARS. More than 7 in. above normal.	GOOD YEARS. More than 2 in. above normal.	NORMAL YEARS.	BAD YEARS. Less than 2 in. below normal.	VERY BAD YEARS. Less than 7 in. below normal.
4	7	4	5	3

APPENDIX C.

MAZOE RIVER AT THE POORT.

DISCHARGE FIGURES 1917-18.

Date.	Normal Flow.	Total Flow.	Storm Flow.	Average Rainfall on Catchment.	Discharge Storm Flow.	Maximum Rainfall recorded in 24 Hours.
	Acre-Feet	Acre-Feet.	Acre-Feet.	Inches.	Per Cent.	Inches.
1-31 July . .	78	78	..	Nil	Nil	..
1-31 August .	46	46	..	0·25	Nil	0·11
1-30 September	30	30	..	0·03	Nil	0·4
1-31 October .	Nil	Nil	..	0·15	Nil	0·15
1-30 November.	140	7,740	7,600	5·59	19·61	2·63
1-31 December.	250	21,308	21,058	10·19	29·80	4·62
1-31 January .	500	13,336	12,836	12·13	15·44	2·00
1-28 February .	1,200	45,192	43,992	10·97	57·81	2·09
1-31 March . .	1,300	13,112	11,812	6·71	25·38	2·21
1-30 April . .	1,800	5,280	3,480	0·81	61·96	0·64
1-31 May . .	1,400	2,418	1,018	0·42	35·00	0·26
1-30 June . .	1,200	1,200	..	Nil	Nil	..
		109,740		47·25		

Percentage discharge during 1917-18 (inclusive of normal flow), 37·5.

Note—The discharges were not measured directly in cases of heavy floods, as no means of observing with greater accuracy were available.

The flood-discharges were estimated and are probably excessive. The highest flood in the known history of the river occurred on the 13th February, at the Poort, when a peak discharge of 10,130 cubic feet per second, of 5 hours' duration, was recorded.

APPENDIX D.

MAZOE RIVER AT THE POORT.

DISCHARGE FIGURES 1918–19.

Date.	Normal Flow.	Total Flow.	Storm Flow.	Average Rainfall on Catchment	Discharge Storm Flow.	Maximum Rainfall recorded in 24 Hours at any Station.
	Acre-Feet.	Acre-Feet.	Acre-Feet.	Inches.	Per Cent.	Inches.
1–31 July . .	1,054	1,054	..	Nil	Nil	..
1–31 August .	868	868	..	Nil	Nil	..
1–30 September.	600	600	..	Nil	Nil	..
1–31 October .	300	341	41	0·44	1·34	0·60
1–16 November .	112	112	..	0·93	Nil	1·60
17–24 November	56	818	762	1·94	5·67	1·70
25 November–7 December .	91	1,714	1,623	4·14	4·65	1·94
8–31 December .	192	7,009	6,817	6·52	15·08	3·40
1–19 January .	190	12,954	12,764	6·84	26·91	1·94
20 January–10 February .	420	16,530	16,110	7·38	31·48	1·95
11–28 February .	540	16,210	15,670	7·48	30·21	2·60
1–20 March . .	1,200	4,608	3,408	1·54	31·92	1·52
21–31 March .	1,218	1,218	..	Nil	Nil	..
1–30 April . .	1,818	1,838	20	1·06	0·27	0·55
1–31 May . .	717	717	..	Nil	Nil	..
1–30 June . .	690	690	..	Nil	Nil	..
		67,281		38·27		

Percentage discharge during 1918–19 (inclusive of normal flow), 25·36.

Note.—The actual discharges were measured directly in the river-channel at the Poort.

APPENDIX E.

MAZOE RIVER AT THE POORT.

DISCHARGE FIGURES 1919-20.

Date.	Estimated Normal Flow.	Total Flow.	Estimated Storm Flow.	Average Rainfall on Catchment.	Discharge Storm Flow.	Maximum Rainfall recorded in 24 Hours at any Station.
	Acre-Feet.	Acre-Feet.	Acre-Feet.	Inches.	Per Cent.	Inches.
1-31 July . .	861	861	..	..	Nil	..
1-31 August .	741	741	..	..	Nil	..
1-30 September	450	450	..	..	Nil	..
1-31 October .	310	330	20	1·25	0·28	0·77
1-20 November	179	423·70	244·70	2·25	1·59	1·54
21-27 November	62·33	196·08	133·75	1·17	1·64	1·01
28 November-8 December .	78·50	180·41	101·91	0·43	3·42	0·61
9 December-30 December	194·80	497·41	302·61	4·48	0·98	4·45
31 December-7 January .	69·91	69·91	..	0·09	Nil	0·52
8-10 January .	27·00	52·01	25·01	0·08	4·50	0·32
11-14 January .	36·00	127·30	91·30	1·52	0·86	1·90
15-20 January .	54·00	222·18	168·18	2·09	1·16	1·85
21-31 January .	99·00	803·77	704·77	4·52	2·25	4·35
1-7 February .	Not determined.	935·57	935·57	2·54	5·31	3·15
8-17 February .	" "	1,614·08	1,614·08	3·88	6·00	3·00
18-23 February	" "	1,257·74	1,257·74	2·17	5·91	1·65
24-29 February	" "	2,901·62	2,901·62	3·60	6·82	3·33
1-17 March .	3,704·00	5,004·26	1,300·26	1·10	17·04	1·01
18-20 March .	460·00	2,096·09	1,636·00	1·20	19·66	1·23
20-31 March .	1,802·00	2,402·06	600·06	0·82	10·55	0·26
1-8 April . .	700·00	1,129·00	429·00	0·87	7·11	0·85
9-30 April . .	1,100·00	1,422·00	320·00	0·32	10·52	0·42
1-4 May . .	160·18	190·20	30·02	0·23	2·00	0·20
5-31 May . .	1,073·00	1,073·00	..	0·32	Nil	0·32
1-30 June . .	848·00	848·00	..	..	Nil	Nil
		25,828·90		34·93		

Percentage discharge during 1919-20 (inclusive of normal flow), 10·68.

Note.—The discharges from July to September were measured directly in the river-channel at the Poort.

From October to July the stored water in the dam was recorded; this does not allow for absorption taking place in the reservoir-bed.

APPENDIX F.

CLEVELAND DAM EVAPORATION DATA.

DAILY EVAPORATION LIMITS: INCH.

	1913-14		1914-15		1915-16		1916-17		1917-18		1918-19		1919-20		Average 1913-20	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	M	Max.	Min.	Max.	Min
July . .	0·27	0·13	0·21	0·02	0·21	0·11	0·24	0·16	0·29	0·16	0·27	0·13	0·27	0·16	0·25	0·12
August . .	0·30	0·14	0·35	0·07	0·29	0·19	0·32	0·13	0·29	0·16	0·24	0·16	0·32	0·16	0·31	0·14
September .	0·48	0·10	0·40	0·24	0·40	0·06	0·40	0·19	0·35	0·27	0·40	0·24	0·40	0·21	0·40	0·19
October . .	0·53	0·24	0·45	0·29	0·45	0·20	0·53	0·27	0·45	0·29	[1]	[1]	0·49	0·11	0·49	0·23
November .	0·43	0·12	0·59	0·09	0·51	0·13	0·51	0·07	0·45	0·02	0·44	0·11	0·43	0·12	0·48	0·09
December .	0·53	0·01	0·34	0·03	0·35	0·00	0·32	0·05	0·29	0·02	0·43	0·14	0·37	0·08	0·38	0·05
January . .	0·40	0·12	0·35	0·01	0·35	0·02	0·37	0·01	0·30	0·01	0·40	0·03	0·40	0·16	0·37	0·05
February . .	0·32	0·03	0·35	0·00	0·40	0·14	1·29	0·11	0·35	0·04	0·53	0·00	0·37	0·08	0·52	0·06
March . . .	0·35	0·19	0·35	0·00	0·41	0·09	0·32	0·07	0·32	0·08	0·32	0·07	0·37	0·12	0·35	0·09
April . . .	0·32	0·11	0·29	0·03	0·32	0·05	0·29	0·10	0·40	0·16	0·35	0·13	..	..	0·33	0·10
May . . .	0·27	0·13	0·32	0·13	0·32	0·08	0·29	0·11	0·29	0·13	0·32	0·11	..	..	0·30	0·12
June . . .	0·24	0·05	0·21	0·13	0·21	0·08	0·27	0·16	0·27	0·13	0·24	0·13	..	..	0·21	0·11

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[1] Evaporameter undergoing repairs.

APPENDIX F—*continued.*

CLEVELAND DAM EVAPORATION DATA.

MONTHLY TOTAL EVAPORATION: INCHES.

	1913 14	1914 15	1915 16	1916 17	1917 18	1918 19	1919-20	Average 1913-20
July . .	5·64	4·94	5·39	6·11	5·89	5·71	6·28	5·71
August .	5·71	6·48	6·92	6·91	6·66	6·06	7·78	6·95
September	8·96	9·35	8·61	9·74	9·17	9·29	9·29	9·20
October .	12·72	12·07	11·26	12·17	11·63	[1]	9·78	11·60
November	9·32	9·98	8·99	7·29	8·40	9·56	8·77	8·90
December .	11·81	7·47	7·89	6·69	5·27	8·66	8·55	8·05
January .	8·60	6·63	6·41	7·53	4·82	6·46	8·20	6·95
February .	4·86	5·74	8·72	7·73	5·16	5·73	7·19	6·59
March . .	9·14	7·44	7·99	7·57	7·33	8·12	6·93	7·79
April . .	7·81	5·98	7·16	6·39	7·10	8·03	..	7·09
May . .	5·87	6·80	6·57	6·12	6·63	7·48	..	6·58
June . .	4·82	5·02	5·26	6·15	5·56	5·48	..	5·38
Annual Total	97·06	87·90	91·17	90·40	83·62	..	..	90·78 Mean Annual Evaporation 1913-20

90·03

Mean Annual Evaporation 1913-18.

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[1] Evaporameter undergoing repairs.

APPENDIX G.

CEMENTATION DATA.

Hole.	Depth of Drill.	Depth of Ironstone Drilled.	Depth of Felsite Drilled.	Cement.	Remarks.
	Feet.	Feet.	Feet.	No. of Bags.	
A1	12½	8	4½	51	
A2	17	11	6	32	
A3	12½	10	2½	49	
A4	25	21	4	31	
A5	29½	27	2½	42	
A6	28½	27	1½	48	
A7	32	31	1	89¼	
A8	51	47	4	5¾	
A9	52	..	..	3	Suspended after first treatment.
A10	20	..	..	33¼	
B1	14	12¼	1½	27¼	
B2	15½	12½	3	86¼	
B3	17	13½	3½	18¼	
B4	25	23	2	25¾	
B5	26	25	1	151	
B6	38½	36½	2	26¾	
B7	41	38½	2½	27	
B8	54	52	2	29	
B9	66	63	3	81	
B10a	42	40	2	..	Abandoned.
B11	72	69	3	373	
B11a	28	28	0	111	
B12	79	77	2	163	
C1	42	10	32	31	Experimental holes.
C2	23	16½	6½	55	
C3	16	13	3	34¼	
C4	23	20	3	134½	
C5	26	24	2	32	
C6	29	27	2	20	
C7	44	42	2	36	
C8	37	37	0	39¾	Suspended after first treatment.
C8a	51	49	2	30	
C9	60	58	4	3⅝	
C9a	72	70	2	3	
C11	74	70	4	68	
C12	90	87	3	52½	
C13	96	95	1	56	
C14	92	89	3	88	
D1	5½	4	1½	0	Refused to take grout at 800 lbs. per sq. in. pressure
D2	5½	4	1½	5¼	
D3	18½	14	4½	18	
D4 •	23	15	8	34¼	12-inch deep fissure at 20-foot depth.

APPENDIX G—*continued.*

CEMENTATION DATA.

Hole.	Depth of Drill.	Depth of Ironstone Drilled.	Depth of Felsite Drilled.	Cement.	Remarks.
	Feet.	Feet.	Feet.	No. of Bags.	
D5	25	21½	3½	179	
D6	25	23½	1½	34	
D7	27	23	4	57½	
D8	32	26½	5½	322	
D9	35½	32	3½	6	
D10	22	22	5	..	Abandoned.
E1	5½	4	1½	10¼	
E2	14	7½	6½	16¾	
E3	23	17	6	26½	12-inch deep fissure at 20-foot depth.
E4	25½	20	5½	25½	
E5	28	26	2	7	
E6	27½	26	1½	107	
E7	24	21	3	112	
E8	50	46	4	12	
E9	73	71	4	4	
E10	79	76	72	72	
E11	85	84	45	45	
F1	5½	4	1½	4¼	
F2	14	9	5	30¼	
F3	18½	14	4½	51	
F4	27½	23	4½	47½	Hole duplicated.
F5	28½	26	2½	150	
F6	30	30	2½	81	
F7	33½	30	3½	41¼	
F8	48	45	3	14½	
F9	49½	47	2½	3	
F10	53	50	3	44	
F11	57	55	2	41	
F12	70	69	1	16	
F13	80	88	2	25	
F14	..	..	..	29	
X1	8	8	..	2¼	
X2	8	8	..	2¼	
X3	25	25	..	25¼	
X4	30	30	..	123¼	
X5	25	25	..	8¼	

APPENDIX H.

COST OF CEMENTATION.

	Quantity.	Rate.	Amount.	Total.
1. Clearing Ground :—			£ *s.*	£ *s.*
(*a*) Vegetation, soil	2,220 cub. yds.	1*s.*	111 0	
(*b*) Covering Ironstone for platforms	1,050 ,, ,,	3*s.*	157 10	
				268 10
2. Drilling (labour, fuel, oil, etc.) :—				
(*a*) "H" Sullivan Drill . .	2,000 lin. ft.	8*s.*	800 0	
(*b*) "D" Beauty Drill . . .	1,666 ,, ,,	7*s.* 6*d.*	624 15	
(*c*) Rock Drills (percussion) .	60 ,, ,,	5*s.*	15 0	
				1,439 15
3. Carbons (Brazilian) :—				
(*a*) "H" Drill	62 carats	£14 0	868 0	
(*b*) "D" ,, ...	57 ,,	£16 10	940 0	
				1,808 0
4. Cement Grout :—				
(*a*) Cement (188 lbs. per bag) .	3,868 bags	18*s.*	3,481 14	
(*b*) Pumping	..	..	1,099 16	
				4,581 10
5. Plant :—				
(*a*) Cost of transport and erection	..	..	..	1,700 0
6. Fees, Salaries, Travelling, etc. .	..	..	..	2,190 0
Gross total cost	..	..	..	11,987 15
Deduct value of realizable assets	..	..	..	1,987 15
Net total cost	..	..	..	£ 10,000 0

UNIT COSTS.

25,000 cubic yards of rock treated . . .	8*s.* per cubic yard.
3,666 linear feet of cementation in holes	£2 14*s.* 6*d.* per foot (drilling included).
3,666 ,, ,, drilling only . . .	£1 5*s.* per foot.

APPENDIX J.

ABSTRACT OF COST OF DAM.

Item of Work.	Quantity.	Rate.		Amount.		Total.	
1. Dam-Foundations :—		s.	d.	£	s.	£	s.
(a) Preparatory excavation — loose soil and ironstone .	3,500 cub. yds.	2	6	475	0		
(b) Excavation of adopted site	9,000 ,, ,,	3	0	1,350	0		
						1,825	0
2. Concrete :—							
(a) Cement (188 lbs. per bag).	55,000 bags	18	0	49,500	0		
(b) Concrete blocks, making and placing (60,000) .	3,900 cub. yds.	22	0	4,290	0		
(c) Concrete mixing and placing	17,400 ,, ,,	1	0	870	0		
(d) Stone-crushing, hauling, and quarrying . . .	25,000 ,, ,,	5	6	6,875	0		
(e) Sand ballasting; hauling and washing	13,000 ,, ,,	2	9	1,787	10		
						63,322	10
(Unit concrete costs) . .	21,100 ,, ,,	60	0	..		..	
3. Ironwork in Dam	..	..		4,625	0	4,625	0
4. Miscellaneous :—							
(a) Wagon-road construction	..	..		350	0		
(b) Light railway-track, including hire of rails. .	..	..		2,750	0		
(c) Buildings	..	..		2,420	0		
(d) Firewood	..	..		1,200	0		
(e) Plant	..	..		9,578	0		
(f) Erection and maintenance of plant	..	..		2,644	0		
(g) Insurance and medical .	..	..		782	0		
(h) Influenza epidemic. . .	..	..		274	0		
						19,998	0
5. Cementation per Appendix (H)	..	..		11,987	15	11,987	15
6. Land, legal, surveys, engineers' salaries, fees, and general office and headquarters expenses .	..	..		37,500	0	37,500	0
						139,258	5
Less value of saleable buildings, plant, carbons, etc.						7,000	0
Net cost of dam . . .						£ 132,258	5

APPENDIX K.

DISCHARGES: CUBIC FEET PER SECOND.

1919-20.	Average Water-Level in Dam.	Number of Main Leaks.												Total of Leakage Discharge.
		1	2	3	4	5	6	7	8	9	10	11	12	
October .	4078·00	0·01	..	..	0·05	..	..	..	..	..	..	..	..	0·15
November .	4085·00	0·02	..	0·04	0·05	0·02	..	..	..	..	..	..	..	0·20
December .	4090·00	0·03	0·10	0·05	0·05	0·02	0·04	..	..	..	..	..	..	0·30
January .	4110·00	0·04	0·12	0·07	0·06	0·02	0·04	0·01	..	..	..	0·05	0·10	0·50
February .	4115·00	0·05	0·12	0·10	0·10	0·10	0·10	0·05	0·05	0·10	0·10	0·20	0·70	1·94
March . .	4125·00	0·05	0·15	0·15	0·15	0·15	0·15	0·15	0·20	0·20	0·20	0·55	1·05	3·20
April . .	4129·00	0·05	0·15	0·14	0·16	0·10	0·14	0·19	0·23	0·18	0·20	0·70	1·50	4·20
May . . .	4130·00	0·05	0·13	0·13	0·17	0·10	0·13	0·13	0·20	0·13	0·15	0·70	1·40	3·80
June . .	4130·70	0·06	0·12	0·12	0·11	0·10	0·10	0·12	0·15	0·10	0·15	0·65	1·30	3·50
July . .	4130·00	0·06	0·11	0·11	0·10	0·10	0·15	0·11	0·10	0·05	0·15	0·60	1·20	3·20

(*Paper No. 4457.*)

"The Works for the Augmentation of the Supply of Water to the City of Capetown, South Africa."

By DAVID ERNEST LLOYD-DAVIES, M. Inst. C.E.

IN 1913 the principal municipalities situated within the boundaries of the peninsula of the Cape of Good Hope, consisting of Capetown, Sea Point, Woodstock, Mowbray, Rondebosch, Claremont, and Kalk Bay, combined, under the City of Capetown Unification Ordinance, to form the City of Greater Capetown. The main objects of the unification of these districts under one administrative body were the augmentation of the water-supply to the incorporated area, and the sewerage of the various townships now known as the southern suburbs of the city. The shortage of water during the summer months had become very acute, and the newly-formed Council decided, notwithstanding the outbreak of war in 1914, that it was necessary to deal with this service, so essential to the well-being of the community, without delay.

Before unification Capetown proper and Sea Point were supplied both from the Woodhead and Hely-Hutchinson reservoirs, situated on Table Mountain, which had a combined storage-capacity of 410 million gallons, and from local springs on the mountain slopes. The southern suburbs obtained their supply from the Albion and the Kommetje springs, together with a storage-reservoir at Newlands. of 30 million gallons capacity, under the jurisdiction of a water board. Kalk Bay relied upon a small storage-reservoir on the mountain and a very limited supply from springs. The combined yield from these sources during a dry summer amounted to 4,750,000 gallons per diem, a precarious supply in a semi-tropical climate for a population, including visitors, of nearly 200,000, and for the requirements of the State railways and shipping. (The present average summer consumption, now that the augmented supply is available, exceeds 7 million gallons per diem.) The seriousness of the position is disclosed by the fact that it was necessary to cut off the supply entirely for 1,280 hours during 1916, 2,199 during 1917, 1,810 during 1918, and 2,218 during 1920, and to prohibit the use of fresh water for gardens and for watering the streets during the summer. During the latter part of the exceptional drought experienced in the season of 1919–20 the situation was so critical that it was imperative to restrict the supply for 20 hours daily—a deplorable state of affairs.

Towards the end of 1915 the City Council appointed a board of engineers, consisting of Messrs. W. A. P. Tait and Thomas Stewart, MM. Inst. C.E., and the Author, to examine and report upon the various sources available for the augmentation of the water-supply. The reference necessitated exhaustive inquiry into the eligible schemes within a radius of 50 miles from the city. After receiving the final recommendations of the Board, the Council decided, early in 1917, to embark upon an entirely new supply from the Steenbras source, and instructed the Author to undertake the design and execution of the scheme with all possible dispatch. The authority of the enrolled voters within the unified area to raise the loan to finance the undertaking was obtained on the 25th July, 1917, and the preparatory work was sufficiently advanced to enable the order for the cast-iron pipes to be placed in England by cable forthwith.

The Steenbras Scheme.—The catchment-area, 23½ square miles in extent, is about 40 miles distant from the centre of the city, and ranges in elevation from 1,050 to 4,150 feet above sea-level. It is drained by the Steenbras and Kogel Berg rivers, the confluence taking place in a natural basin eminently suitable for the formation of a large storage-reservoir by the erection of a wall across the comparatively narrow neck through which the main stream originally flowed. From the neck the water falls 1,000 feet, through a gorge ¾ mile in length, directly into the sea. The Council have either acquired or obtained jurisdiction over the whole of the catchment-area, besides the land on both sides of the river as far as the coast, and they possess the whole of the water-rights, thereby avoiding any complication in regard to compensation-water. All the farms and buildings in the valleys have been vacated, and the afforestation of the upper reaches has been commenced, about 500 acres being already planted with pine-trees.

Owing to the rocky nature and steepness of the slopes of the mountains surrounding the catchment-area, the discharge is very rapid. The daily average river-flow has been measured over a 60-foot weir constructed below the site of the dam and registered up to 1920 by a recorder. The rainfall and the daily average river-flow since 1915 have been as follows :—

Year.	Rainfall.	Daily Average River-Flow.	Ratio of River-Flow to Precipitation.
	Inches.	Gallons.	Per Cent.
1916	49·57	31,401,000	68
1917	49·59	35,110,000	76
1918	43·90	31,712,000	77
1919	38·23	23,959,000	67
1920	48·67	37,972,000	84
1921	40·28	25,536,000	68

The approximate average valley temperature for the year is 64·5° F. There was an exceptional drought during 1919, and it is estimated that the yield from the watershed will be 25 million gallons per diem in ordinary dry years. The annual evaporation recorded in the valley during 1922 amounted to 80·8 inches.

After examination of local statistics the Council were advised that a further supply of 5 million gallons per diem beyond that already obtained from the peninsula sources would probably satisfy the demands of the city for a period of 10 years, and that it was possible to complete the work within 3 years in normal times. It was eventually decided to adopt this figure for the first section of the scheme, owing both to anticipated difficulties in obtaining material and labour, as a result of dislocation brought about by the war, and to the high rate of interest on loans, the time for completion being the predominant factor. The main features of the scheme are the storage-reservoir and dam in the Steenbras valley, the tunnel through the Hottentots Holland range, and the pipe-line from Steenbras to Capetown (Fig. 1, Plate 6). The construction of the dam and tunnel in the first instance was entrusted to contractors, and the laying of the pipe-line was undertaken by direct labour.

Storage-Reservoir and Dam.—The reservoir-capacity required to balance the discharge from the watershed for a daily supply of 5 million gallons during a normal dry year amounts to about 600 million gallons, and the wall (Figs. 2, 3, and 4, Plate 6) is designed to impound this volume. The ultimate storage-capacity of the basin is about 6,000 million gallons. In the preliminary design the construction of a dam of sufficient strength to permit of its height being increased to meet future demands for increased storage was discussed, but it was evident that it would not be possible to complete such a structure in the time stipulated, under such abnormal conditions. Further, it was proved that the extra interest and sinking-fund on the larger proposal over a period of 10 years would exceed the total cost of the smaller dam necessary to meet present needs.

The site finally chosen is situated slightly higher up the neck than that originally selected, leaving the most favourable position free for a larger wall in the future. The dam, arched in shape, is built upon a foundation excavated in sandstone of the Table Mountain series, the greatest depth of the trench being 20 feet. The principal dimensions are: length, 900 feet; maximum height, 62 feet; maximum width at the base, 38 feet, tapering to 8 feet 6 inches at the crest; and length of the spillway, 240 feet. It is built of Cyclopean masonry, the plums being embedded in 6-to-1 concrete, and is faced on both sides with concrete blocks. During heavy rainfall the river rises very rapidly, and adequate provision had to be made to cope with floods up to 1,000 million gallons per 24 hours. During the first stage of construction the river was conveyed over the trench by means of a large flume. When the work reached the level of the river-bed, a free way, 8 feet by 6 feet, which was calculated to pass average floods, was left through the base of the wall. The excess during extraordinary storms was allowed to flow over the crest, the stream being confined as far as possible to definite channels well away from green work.

The contract was signed in December, 1917, the time for completion being 2 years. Owing to the many difficulties due to the war, the contractor had only been able to excavate about two-thirds of the trench and to commence the masonry foundation by the middle of 1919; and at that stage the Council decided to proceed with the work by direct labour. The sluice-gate commanding the free way was closed in October, 1920, in time to impound the full quantity of water required for the summer season, and the wall was completed in April, 1921, 19 months after the date when it was taken over from the contractor.

Hottentots Holland Tunnel.—In order to convey the water from the reservoir to the commencement of the pipe-line above the village of Gordon's Bay (Fig. 1) a tunnel, 2,648 feet in length, had to be driven through the range of hills skirting the border of the Steenbras valley. The sandstone through which the tunnel is driven varies in nature from soft white friable stone to the hardest quality found in the Table Mountain series. For more than half the drive from the Steenbras end the rock was soft and, except for occasional fissures, easily drilled. These fissures, from which sand and water issued at intervals, caused delay and added considerably to the cost of the work. The drive from the Gordon's Bay face passed through harder rock and gave less trouble. In May, 1920, when the drives were about 150 feet apart (Figs. 5, 6, and 7, Plate 6), exceptionally large fissures up to 8 feet in width were encountered in the heart of the mountain, and periodical rushes of water and slime were experienced, which completely choked the tunnel for a distance of several hundred feet each time they occurred. It was not possible to obtain in South Africa adequate plant to deal with such an unexpected emergency, and the difficulty appeared at one time to be almost insurmountable. It was eventually overcome by approaching gradually from each face by means of a series of substantial bulkheads, 6 months after the fissures were struck. Through the soft rock, from the Steenbras face to the centre, the tunnel is lined with cement concrete (Fig. 9, Plate 6), but the portion in hard rock from the centre to the outlet is provided with a dished concrete invert only. The concrete section under the fissures was heavily reinforced (Figs. 6, 7 and 8). As in the case of the dam, the contractor failed to make satisfactory progress with the work, and it was undertaken by direct labour after 600 feet had been driven. The work was completed on the 5th January, 1921, and water passed through 10 days afterwards.

Pipe-Line.—From the portal of the tunnel the water passes through 24-inch and 18-inch cast-iron pipes to the upper and lower break-pressure tanks. Both these tanks contain 50,000 gallons of water when full. From the lower tank the cast-iron pipe-line, 30 inches in diameter, starts along the slopes above Gordon's Bay village and continues to the main road to Somerset Strand, the pipes being carried over the ravines by steel troughing resting on masonry piers. The line then takes the direction of the Strand, Lourens river and de Beer's roads, as far as the Capetown-Caledon main road, which it follows for the remaining distance through Kuils river, Belleville, and Maitland to the Capetown service-reservoirs. The pipe-line on its way to Capetown passes under the Lourens, Eerste, and Liesbeek rivers, and under the permanent way at Stikland and the Salt river railway-junction. At the latter point the pipes are laid in a concrete culvert constructed by the Railway Administration.

The water-level of the lower break-pressure tank is 352·05 feet above sea-level, and the water-level at the lower service-reservoir is 189·44; the length of the pipe-line between these points is 39 miles, 2,161 feet. The inlet level at the Molteno service-reservoir, allowing 4 inches for the depth of flow over the weir, is 312·6, and the distance from the lower break-pressure tank to the Molteno reservoir is 39 miles, 2,737 feet. The test values of the pipe-line, measured by Venturi meters after it has been in service off and on for 20 months, are 6,780,000 gallons per diem when discharging at the lower reservoir and 3,400,000 gallons per diem at the Molteno reservoir. Some slight difficulty due to air is experienced in the main when testing the discharge into the lower service-reservoir.

The maximum static head on the line is 352 feet, and the pipes range from $1\frac{3}{16}$ inch to $\frac{27}{32}$ inch in thickness, in accordance with the maximum pressure they have to withstand. The average weight of each pipe is just under 2 tons, and the total weight of the pipe-line amounts to 33,802 tons. The loss due to pipes damaged through handling and during transit from the pipe-yards in Great Britain to the trench was 0·647 per cent. The light pipes, $\frac{27}{32}$ inch in thickness, carried badly, and, after the first few consignments had been landed, a minimum thickness of $\frac{15}{16}$ inch was imposed. Some of the consignments, of various thicknesses, arrived without wrought-iron rings shrunk on the spigots, and the percentage of damaged pipes amongst them was no greater than amongst those which arrived with protecting rings.

The sluice-valves, 21 inches in diameter, with taper pipes, are fitted with a 6-inch by-pass, which serves to prevent shock when the valves are operated. Air-valves are fixed on all rises. Self-acting throttle-valves, which close when a burst occurs, are placed at points about one-third and two-thirds of the length of the line, and reflux-valves are included at the Capetown end of the main. The lower break-pressure tank contains a special automatic valve to control the inlet to the pipe-line.

The first pipe was laid in February, 1919, and the last pipe was jointed up on the 7th January, 1922, the largest number of pipes laid by a single gang in a day being fifty-three. On the whole, the nature of the ground through which the pipe-line is laid was favourable, but certain lengths were exceptionally bad, particularly in the neighbourhood of Stikland, where running sand was encountered, necessitating the use of heavy timber. The complete pipe-line was tested to 10 per cent. in excess of the working-pressure, and forty-one defective or damaged pipes had to be replaced owing to bursts.

COMPARISON BETWEEN ACTUAL AND COMPUTED DISCHARGES.

The diameter of the Steenbras pipes is 30 inches, and the net hydraulic gradient of the 39½-mile siphon leading to the Molteno reservoir is 1 in 5,364. Owing to delay in the arrival of the Venturi meters, the pipes had already been in use for 9 months before the first meter-reading was made on the 7th December, 1921. Nevertheless, the results of inspection and the meter-records themselves indicated that the pipes were still new and clean in the sense used by the compilers of formulas; but there is evidence of a diminution of the discharge during 1922 amounting to 4 or 5 per cent.

The actual discharges into this reservoir in 1922 were as follow:—

	Gallons per day.
Beginning of year	3,560,000
Mean of year	3,460,000
End of year	3,400,000

For the purpose of comparison the discharge has been computed by the following recent or well known formulas:—

(1) Barnes (new pipes) $v = 174\cdot1\ m^{0\cdot709}\ i^{0\cdot529}$

(2) Santo Crimp $v = 124\ m^{0\cdot67}\ i^{0\cdot6}$

(3) Williams and Hazen $v = 171\cdot1\ m^{0\cdot63}\ i^{0\cdot54}$

(4) Mallett[1] $v = C\sqrt{mi}$, in which

for (*a*) new pipes $C = \dfrac{172}{1 + 30\dfrac{n}{\sqrt{m}}}$, $n = 0\cdot011$

„ (*b*) encrusted pipes $C = \dfrac{162}{1 + 30\dfrac{n}{\sqrt{m}}}$, $n = 0\cdot013$

„ (*c*) pipes badly encrusted $C = \dfrac{128}{0\cdot8 + 32\dfrac{n}{\sqrt{m}}}$, $n = 0\cdot015$

„ (*d*) pipes very badly „ $C = \dfrac{112}{0\cdot6 + 52\dfrac{n}{\sqrt{m}}}$, $n = 0\cdot017$

The results are:—

		Gallons per day.
(1) Barnes	Discharge	= 3,422,000
(2) Santo-Crimp	„	= 3,280,000
(3) Williams and Hazen	„	= 3,267,000
(4) Mallett:—		
(*a*) New	„	= 3,473.000
(*b*) Encrusted	„	= 3,105,000
(*c*) Badly encrusted	„	= 2,603,000
(*d*) Very badly encrusted	„	= 1,866,000

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[1] Minutes of Proceedings Inst. C.E., vol. ccviii, p. 10.

Formula (b) with $n = 0{\cdot}013$ is considered by Mr. Mallett to be applicable to waters giving minimum incrustation. Formulas (c) and (d) are to be used for soft moorland waters, according to the age of the main. Formula (b) may be used for acid waters, passing into (c) after about 20 years. This allows for a loss of 25 per cent. in that time. *Fig. 10* (p. 347) shows that the deterioration formula adopted for the Steenbras siphon happens to give the same percentage in 20 years.

(5) From Mr. G. M. Lawford's Table[1] of C in the formula $v = C\sqrt{mi}$. Taking $v = 1{\cdot}34$ (the actual velocity in the pipes as recorded at the beginning of 1922), $C = 121{\cdot}5$, and the discharge = 3,477,000 gallons per day.

Loss of Discharge through Incrustation.—Two Papers dealing with this question were discussed by The Institution in February, 1919.[2] Mr. Barnes there proposed, for the velocity of water in clean asphalted pipes, the formula

$$v = 174{\cdot}1\ m^{0{\cdot}769}\ i^{0{\cdot}529} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

which is of the Hagen type. To this he added another for the percentage loss of discharge after use, namely,

$$e = 13\ t^{0{\cdot}37} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which e denotes the percentage loss of discharge compared with equation (1), and t denotes the time of use in years. The discussion and correspondence clearly established the fact that, in the form given by Mr. Barnes, based solely on the data supplied by the Thirlmere aqueduct (Manchester), the equation was of limited application. Nevertheless, an equation of the same general form, namely,

$$e = pt^q \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

in which the values of the constants p and q are different for different siphons and pipe-lines, will embrace, in fact, the other data brought forward in the course of discussion.

These data are :—

(A) A complete 14-year record and diagram for a portion of the Elan aqueduct, Birmingham, from which a formula

$$e = 6\ t^{0{\cdot}5} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

may be deduced.

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[1] Minutes of Proceedings Inst. C.E., vol. cliii, p. 311.

[2] *Ibid.*, vol. ccviii, pp. 1–67.

(B) A diagram covering a period of 24 years for a 42-inch main, supplied by Professor C. W. L. Alexander from Williams and Hazen's hydraulic Tables. This would give approximately

$$e = 1 \cdot 6\ t^{0 \cdot 9} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

(C) The statements of Mr. W. J. E. Binnie regarding the Coolgardie pipe-line, and of Mr. E. Sandeman with reference to the Burrator main of the Plymouth waterworks, which are respectively consistent with the formulas

$$e = 19 \cdot 0\ t^{0 \cdot 33} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

and

$$e = 18 \cdot 3\ t^{0 \cdot 28} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

In each of the last two cases only two points on the curve are supplied. Equation (6) is derived by taking the average of the percentages mentioned by Mr. Binnie.

Taking the mean of equations (6) and (7), and combining with equation (5), the condition obtains that—

$$p = \frac{1 \cdot 26}{q^{2 \cdot 23}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

or its equivalent

$$q = \frac{1 \cdot 11}{p^{0 \cdot 45}}.$$

Equations (2) (Thirlmere), and (4) (Elan aqueduct) satisfy this condition to a near degree of approximation.

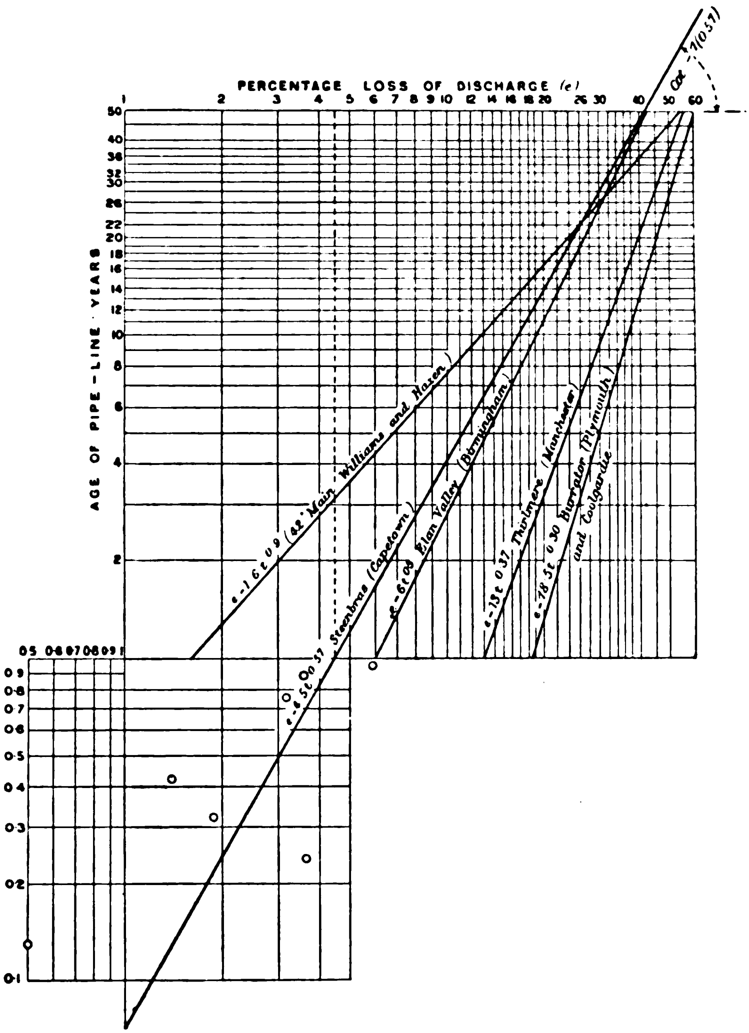
Thus a resulting equation is obtained :—

$$e = \left(\frac{1 \cdot 26}{q^{2 \cdot 23}}\right) t^q \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

in which there is only one constant to be found, namely, q. In the absence of actual records, q would have to be estimated. Its range in the foregoing equations for the different aqueducts will be seen to be from 0·3 to 0·9, with an average of about 0·5—the actual value for the Elan aqueduct. The worse the initial losses of the pipe-line, the lower q will be. The rational interpretation of equation (8) is that incrustation is a process which in the end checks its own rate of progress, and the faster it proceeds at first, the sooner it will begin to slacken its pace. The constant q, therefore, must be a function of p, and vice versa.

Fig. 10 is a logarithmic diagram in which the formulas given previously for various aqueducts are plotted as straight lines. The percentage loss of discharge after any specified period of use may be

Fig. 10.



read at once from the diagram. The formulas on the left of the diagram are applicable to cases where less rapid deterioration is to be expected, and those on the right to more extreme cases. In the absence of definite data the formula for the Elan Valley aqueduct would seem to represent a fair average. It agrees within 2 to 3 per cent. of the mean annual readings shown on the diagram supplied by Mr. Macaulay.[1]

Treatment either of the water or of the pipes may, of course, alter the slope of the line representing the rate of deterioration, and there is some evidence of this in the case of the Elan Valley aqueduct. The formula adopted for the Steenbras siphon is also shown in *Fig. 10*, which is extended downwards in such a way as to include the actual points recorded. Owing to the logarithms being negative, the points appear unduly scattered. *Fig. 11* provides a better means of comparison. Though no great stress can be laid on the exact figures of the Steenbras formula, there is good reason to expect that in its rate of deterioration Steenbras will belong to the left-hand group.

Fig. 11.

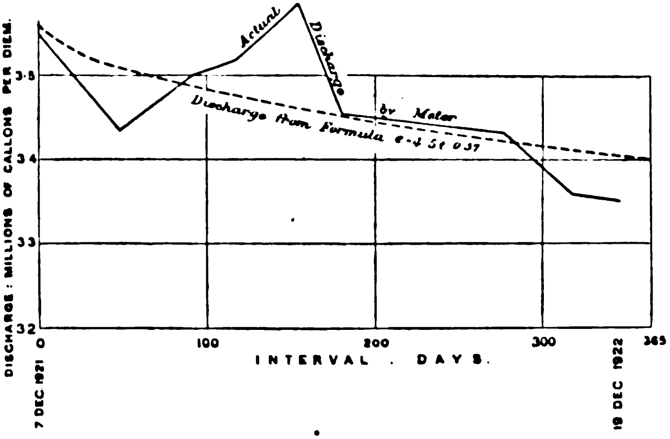


Fig. 12, also logarithmic, provides a graphic solution of the relation between p and q, namely,

$$p = \frac{1 \cdot 26}{q^{2 \cdot 25}},$$

where p denotes the coefficient, and q the index of the power in the general equation

$$e = pt^q.$$

The formulas used are represented on the diagram by small circles. The Steenbras formula, being itself derived from this diagram, is not shown. It should be noticed that the coefficient p is equal to the deterioration at the end of the first year's use, and, this being found in the case of Steenbras to be about 4 or 5 per cent., the diagram shows that the corresponding value of q may be expected to be 0·57. The complete formula reads:—

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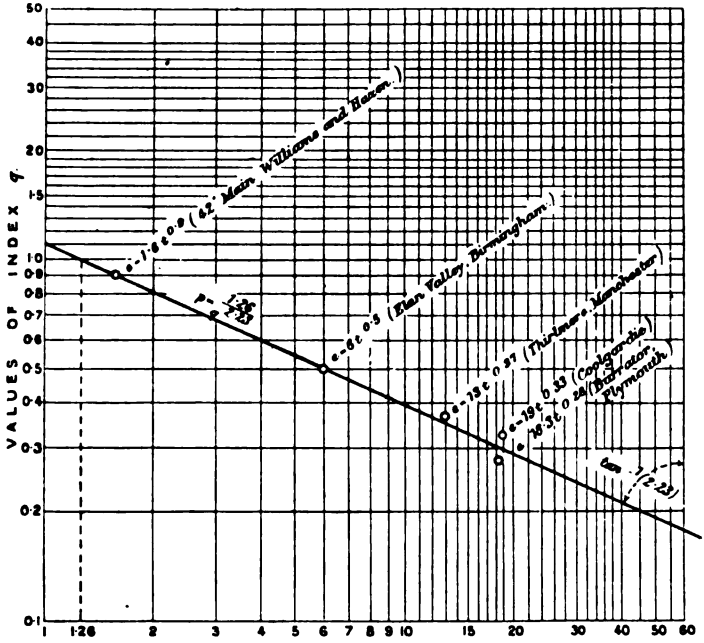
[1] Minutes of Proceedings Inst. C.E., vol. ccviii, p. 25.

$$e = 4\cdot 5\ t^{0\cdot 57}.$$

Fig. 11 shows how this equation applies to the 1922 meter-readings at the Molteno reservoir on the Steenbras pipe-line.

Fig. 12.



Progress of the Works and Expenditure.—Rapid progress was impeded by the loss of valuable time on the contracts, by the interruption due to the influenza epidemic of 1918, and by the labour strike early in 1920. The prohibition of the export of cast-iron pipes, enforced by the British authorities from August, 1918, to January 1919, and the dearth of materials, particularly cement, also caused delay. Despite the abnormal conditions, the scheme was completed in time to supply water to Capetown in January, 1921, and the new works, which involved the expenditure of £1,176,400, were inaugurated by H.R.H. Prince Arthur of Connaught, the Governor-General of the Union of South Africa, on the 9th March, 1921.

Messrs. J. and A. Leslie and Reid undertook to test the cast-iron pipes, and to supervise the design of the specials, on behalf of the Council; and Mr. C. R. Barlow, M. Inst. C.E., organized and supervised the execution of the works, under the direction of the Author, who is indebted to his assistants, Mr. C. Costley White, who prepared the drawings, and Mr. H. C. Mason, for his comparison between actual and computed discharges of cast-iron pipes, which is made use of in the Paper.

The Paper is accompanied by seven sheets of tracings, from some of which Plate 6 and the Figures in the text have been prepared.

Discussion.

The President. The PRESIDENT moved a vote of thanks to the Authors.[1]

Mr. Binnie. Mr. W. J. E. BINNIE, referring to certain particulars given by him of the Coolgardie main, from which Mr. Lloyd-Davies had derived his coefficient, remarked that the Coolgardie main was exceptional, in that the pipes had been exposed to the atmosphere for a long time before being laid, and therefore the coating was partly destroyed. There was a relation between the length of time the pipes had been exposed to the hot sun and the falling-off of discharge due to incrustation. In the case of pipes exposed to the sun for about 6 months the reduction in capacity had amounted to 12 per cent. in 5 years. Where the pipes had been exposed to the action of the sun for about 2 years before being laid, the falling-off had been 44 per cent. The formulas and curves had to be regarded with caution, and, taking the Coolgardie main as an example on which the formula in the Paper was based, it was only right to draw attention to the fact that the conditions were abnormal.

Sir Thomas Ward. Sir THOMAS WARD congratulated the Authors on three valuable Papers, which would form a very useful addition to the "Proceedings." As to how valuable those "Proceedings" might be, he had just had a very interesting example. His colleague on certain dams in north-east Brazil carried about with him wherever he went a complete set of them, and they found them of immense value. Hardly any engineering discussion could be started in an engineers' camp which could not be settled with the aid of those books. The works in north-east Brazil were situated at the western extremity of Gondwana Land, a continent which geologists said once embraced a large part of South America and tropical and Southern Africa, and included a large part of India and all Australia. The Table Mountain dam, as well as the Hobart dam, seemed to be on the margins of that continent. The Federal Government of Brazil were constructing ten large cyclopean-masonry dams, with a total of about 76,000,000 cubic feet of masonry, to store a total of about 6,000,000 acre-feet of water. The foundations had been dug, and concreting had been started on nine of them. One dam was nearly finished. A dam had been built there by a previous generation of engineers—it was said to have been designed by the French hydraulic

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[1] Intimation of the regretted death of Mr. J. C. Ross, which had occurred on the 6th March, was received after the date of this meeting.—SEC. INST. C.E.

Sir Thomas Ward. engineer, Révy. The construction of gravity dams was now being undertaken in all parts of the world. He was interested in the contour map shown in Fig. 2, Plate 5, of Mr. Randall's Paper and in the short paragraph dealing with the canal. Experience of irrigation-works had taught him that their success, once the supply was secured, lay in the proper distribution of the water; and farmers became exceedingly particular as to the accurate alignment and proportioning of their channels. The contour-surveying of broken ground such as that described was an expensive operation in places where there was a great deal of forest growth, and it would be interesting to know how the contours had been obtained, and to have details of the channels that had been designed. In some respects the design of irrigation-channels had deteriorated rather than improved since Mr. R. G. Kennedy published his conclusions. Before that time it was usual to make the channel narrow and deep, with the result that its tail end was usually satisfactory; the trouble then occurred in the upper reaches, where shallower and broader channels with steeper slopes were required. Mr. Kennedy drew attention to the fact that his formulas applied more particularly to the head reaches, and he gave some general rules for the design of the lower sections. But those were not in sufficient detail to enable inexperienced engineers to design with precision. Satisfactory design was still an art which had to be learned by practice and could not all be done by formula. A river, on debouching from the mountains and while rolling along a good deal of silt, was broad and shallow in section with steep bed slopes. When it reached the delta the section became narrow and deep, with easy bed gradients, and he had found in canal practice that that should be imitated. Formulas and diagrams for the better design of irrigation-channels were given in a Paper[1] on Regime Channels, by Mr. E. S. Lindley, M. Inst. C.E.

Mr. Gourley. Mr. H. J. F. GOURLEY remarked that, taking the coefficient for clean pipes in Fidler's formula as 1·76, he calculated that the discharge of the Steenbras pipe-line should be about 3·34 million gallons per day. With the observed discharges the coefficients were 1·715 for 3·56 million gallons and 1·74 for 3·40 million gallons. Those were abnormally low. He asked whether Mr. Lloyd-Davies, in applying the formulas he gave, had had regard to the various diameters of the main. The fact that variation in diameter might have a material effect on the discharge was brought out by an examination of some of the figures for the Birkenhead pipe-line. The section he was dealing with was nominally 19 inches in diameter, but diameters

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[1] Min. Proc. Punjab Engineering Congress, vol. vii (1919), p. 63.

Mr. Gourley. of $18\frac{7}{8}$, $18\frac{5}{8}$, and $18\frac{1}{2}$ inches also occurred. Taking Fidler's coefficient as 1·76, the loss of head in the 16,440 feet of main would be 184 feet when discharging 10 million gallons per day. Taking into account the various diameters of the main, the loss of head would be 203 feet. Alternatively, if the working-head was kept at 184 feet, the discharge was reduced from 10 to 9·55 million gallons per day, a reduction of 4·5 per cent. To ignore the variation in diameter in that case would be to credit incrustation with much more than its true effect. That point was not merely of academic interest. Any known factors should be taken into account in applying a pipe-discharge formula. Other factors—the surface of the metal, the nature of the coating, the care with which the laying had been done, and the bends and other specials introduced—could not well be taken into account in any formula; but in the aggregate and under the worst conditions they might amount to a good percentage. On the Birkenhead pipe-line the filters were at the headworks, and it had been found that, by treating the water and artificially hardening it, there had been apparently no falling-off in discharge during the 2 years the line had been in use. The main consisted largely of cast-iron pipes with a small proportion of steel pipes. The grouting-operations described by Mr. Randall were interesting, and the consequent saving was noteworthy. It seemed to him that, by adopting grouting in preference to excavating in the normal way, £15,000 or £20,000 had been saved. Those operations were directed not only to secure a watertight foundation but also so to strengthen the foundation that the rock would be able successfully to carry loads brought upon it by the dam; and in that respect the work was particularly interesting and suggestive. He had investigated recently the saving which would be brought about by adopting a grouted cut-off under an earthen embankment. Generally the cut-off trench was carried down to considerable depths, for no other reason than to intercept and seal off any fissures or open joints. If, then, the trench were only carried down to such depth as would cut through drift or overlying material, and below that level reliance were placed on drilling and grouting, a considerable saving in costly excavation and concrete refilling would be effected, to say nothing of the saving in time. The case he had investigated was for a dam 1,200 feet long and 125 feet high; and the saving, on a pre-war basis, amounted to £20,000 to £25,000. The successful use of grout in America for similar work suggested that the method was economical both in time and money, and might be adopted with confidence in its permanence and stability. Grouting might be employed in practically all materials except running sand. It had been used to solidify the sand in one tunnel

Mr. Gourley in New York, but the experience was not such as to lead to its adoption in trench work. In most stratified materials the quantity of grout required would be small, or at any rate not excessive. In gravels the quantity required would probably be a good deal more, but even then it might pay to adopt grouting and avoid the very excessive pumping which would follow excavation in such material. In much-fissured and jointed rocks, such as some sandstones, a considerable quantity of grout might be required, and it might be necessary to use a richer mixture and to add sand or other material. If, however, such structure was revealed by trial borings, it was doubtful whether the site would be considered suitable for a dam. In connection with the falling-off in the leakage from the reservoir, and as an example of "self-sealing," he was able to give a few particulars regarding the Carno reservoir at Ebbw Vale, in South Wales. On the left bank there was a horizontal bed of rock, 10 feet thick, very open and jointed, outcropping with its top about 30 feet below the top water-level. When the reservoir was full in November, 1911, the quantity of water that leaked through the bed and was collected below the dam amounted to 560,000 gallons per day. Until March, 1912, the leakage fluctuated more or less with the rainfall from 430,000 to 160,000 gallons per day, the reservoir being full during that period. From March to November, 1912, the leakage fell gradually to 60,000 gallons per day, and it remained practically steady at that quantity at the present time, the effect of the rainfall being scarcely discernible. The water was not wasted; it was collected and brought to the compensation-basin, and paid off as part of the statutory compensation.

Mr. Dalrymple-Hay. Mr. H. H. **Dalrymple-Hay** said he was interested in Mr. Randall's reference to the François cementation process, as he had had some experience of it in the South Yorkshire coal-fields. He asked whether it was the real François cementation process—in which silicate of soda and sulphate of alumina were forced at very high pressure into the ground, in order to lubricate the fissures in the sandstone bed, followed by cement—the only process which allowed the grouting to penetrate the joints in broken rock. The cases he had had to do with were colliery-shafts through new red sandstone rock. The grouting was done under a maximum pressure of about 1,000 lbs. per square inch and had been perfectly successful, but it would not have been successful had not the "silicatization" process been used in conjunction with cement. If money was available, the application of grouting to the ground outside tunnels was a good thing, but it would generally be found to be too costly, and the economical method was to make the joints of a tunnel water-tight in the usual way by caulking, and to spend no money outside.

The President. The **President** said that there was a point of detail in connection with Mr. Ross's Paper on which further information would be useful. Referring to the jointing of reinforced-concrete pipes, Mr. Ross described several methods which had been successful, but they all applied to the jointing of straight pipes laid in line, in which case practically no longitudinal pull was exerted on the joints. He believed that a number of successful joints of this kind were in use in America; but he understood that trouble had arisen in making efficient joints between reinforced-concrete pipes and cast-iron elbows or fittings. In the case of joints between reinforced-concrete pipes and cast-iron bends, a longitudinal stress was exerted on the joints, and this had to be specially provided for. If any members present had had experience in dealing with joints of this kind, the information they could give would be of much interest.

Mr. Alford. Mr. J. S. **Alford** remarked that Mr. Lloyd-Davies stated that the construction of a dam of sufficient strength to permit of its height being increased to meet future demands for increased storage had been abandoned on account of the time which it would have taken to complete the work. That appeared to be a sufficient reason, but the Author added the following statement: "Further, it was proved that the extra interest and sinking-fund on the larger proposal over a period of 10 years would exceed the total cost of the smaller dam necessary to meet present needs." Looking at the matter from a finance point of view there seemed to be something wrong with that statement. He did not see how it was possible to compare an expenditure incurred at the present time with something which would have accumulated at the end of a term of years.

Mr. Dolby. Mr. E. R. **Dolby** said that he had been interested to learn from Mr. Gourley's remarks that the water entering the pipe-lines of the Birkenhead installation had been hardened, and that this hardening effect had left the pipes clean. He presumed Mr. Gourley referred to the action of soft water upon the interior of a cast-iron pipe, producing nodules of iron, and thereby lessening the cross-sectional area of the pipe. He would like to know whether in the Birkenhead installation any method was adopted for de-aerating the water, because it appeared to him that, if the water could be completely de-aerated, the difficulty of a cast-iron accretion inside the pipes would be obviated.

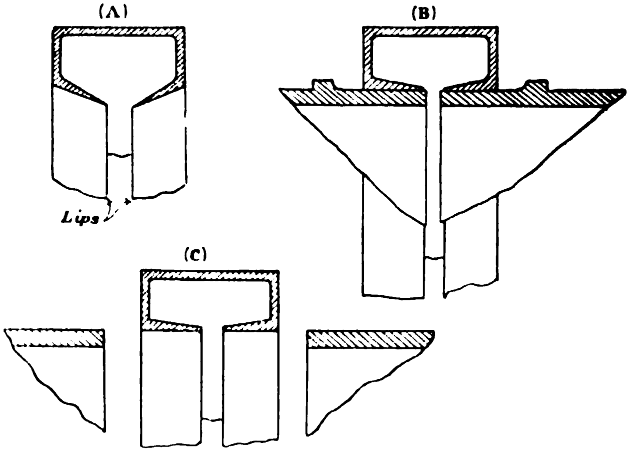
Mr. Binnie. Mr. **Binnie** stated there was no de-aeration process. Hydrated lime was added, and he thought there had been a gradual deposition of carbonate, as the calcium hydroxyl combined with the CO_2 travelling along the pipe. A very fine film was formed over the coating. That film had been traced to a break-pressure tank 27 miles down the line.

Mr. Dolby. Mr. **Dolby** remarked, with regard to the President's question concerning the connection of cast-iron pipes to concrete mains, that he believed Dr. Hele-Shaw's "Victaulic" joint had been used very successfully for such purposes.

Mr. Tudsbery. Mr. M. T. **Tudsbery** stated that he had had some experience with that joint and could speak highly of it from the point of view of its service when movement took place in the pipe-line, its elasticity, and the pressures it would stand. Independent tests carried out within his knowledge had shown the adaptability of the joint to the connection of small pipes laid over very uneven ground. He considered that it would be very suitable for connecting concrete with iron pipes.

Captain Riall Sankey. Captain H. **Riall Sankey** said that a point of interest about the "Victaulic" joint was illustrated in *Figs. 1.* In *Fig. 1*A the flexible ring making the joint was shown before it was put on the pipe; when it was fitted on the pipe the lips were stretched (*Fig.* B), thus making an effective joint, which became tighter as the pressure increased (like a hydraulic leather). Some 50 years ago Messrs. Brotherhood patented a joint-ring as shown in *Fig. 1*C, which did not stretch when placed on the pipes, and therefore no joint was made whether pressure was applied or not. The stretching of the lips made all the difference between a successful and an unsuccessful joint.

Figs. 1.



Mr. Atkins. Mr. M. R. **Atkins** mentioned that he had inspected lately in India a pipe-line in which concrete pipes were used in conjunction with cast-iron pipes, and the connections between the straight concrete

Mr. Atkins. pipes and the cast-iron bends were made with a reinforced-concrete collar and a $\frac{1}{2}$-inch cement mortar joint. The cast-iron specials had spigot ends, and the reinforced-concrete collar was fitted over the cast-iron pipe without difficulty. There were many leaks in the pipe-line, but they were mostly in the joints in the reinforced-concrete portion of the main. The joints over the cast-iron specials had probably been more carefully done and were the soundest of them all.

Mr. Randall. Mr. Randall, in reply, stated that, being engaged on a survey in a remote district of South Africa, he could only refer in general terms to the discussion. He could not, on that account, furnish more detailed information regarding the design of the channels, but in reply to Sir Thomas Ward's question as to the contours he could say that they had been obtained by tacheometrical survey and plotted to a scale of 200 feet to the inch, showing 2-foot intervals on the flat and on gentle slopes, and on more inclined slopes 5-foot intervals. In reply to Mr. Dalrymple-Hay, only water had been used to cleanse the fissures in the rock, and it was applied before, and at the same pressures as, the cement grouting. No other solution of any kind had been used. He thanked the members for their kind reception of his Paper.

⁂ Mr. Lloyd-Davies's reply will be found at p. 369.—Sec. Inst. C.E.

Correspondence.

Mr. Barnes. Mr. A. A. Barnes congratulated Mr. Lloyd-Davies on a very satisfactory solution of the problem relating to the rate of incrustation in cast-iron mains. The fact that it had become possible to find a relation between the two constants p and q in the equation $e = pt^q$ was of interest to engineers concerned with the future capacity of aqueducts which they might have in their charge; for it was realized, when the rate of incrustation for the Thirlmere aqueduct was given[1] as $e = 13\ t^{0 \cdot 37}$, that this equation might be of little use to others who did not happen to have experienced a falling-off of 13 per cent. in the discharging-capacity of their aqueduct at the end of the first year. In fact the discussion at that time produced the somewhat disconcerting information that, whereas one pipe-line had shown only 1·6 per cent. loss after one year's use, another pipe-line had shown as much as 19 per cent.; and it seemed

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[1] Minutes of Proceedings Inst. C.E., vol. ccviii, pp. 1-8.

Mr. Barnes. impossible to reconcile the statements or to obtain much guidance from them. He had, therefore, been agreeably surprised to see in *Fig. 12* of Mr. Lloyd-Davies's Paper that such widely-separated determinants as 1·6, 6, 13 and 18·5 for p were themselves exactly related by means of their respective indices 0·9, 0·5, 0·37 and 0·30 for q. Apparently it was now possible, by making one observation, not only to learn the past history of a pipe-line when the age was known, but also to foretell the future rate of incrustation. Thus, taking the case of the Derwent Valley aqueduct, mentioned by Mr. Sandeman.[1] after 6½ years that aqueduct had lost about 29 per cent. of its discharging-capacity. Hence the equation $e = pt^q$ gave as a first result $29 = p\,(6\tfrac{1}{2})^q$, and secondly, since p and q were now related, $29 = p\,(6\tfrac{1}{2})^{1 \cdot 11 / p^{0 \cdot 45}}$. This equation had only one unknown, and, by solving it, $p = 15 \cdot 9$, and hence $q = 0 \cdot 32$. That meant that the rate of incrustation, past, present, and future, for the Derwent Valley aqueduct was represented by the equation $e = 15 \cdot 9 t^{0 \cdot 32}$. There was a slight risk in basing conclusions upon one observation only, as the period of the year when that observation was made might affect the result. In the cases of both the Elan aqueduct[2] and the Steenbras main (*Fig. 11*), there was strong evidence of a greater discharge in summer than in winter, so that probably the mean of a consecutive summer and winter reading would give a fairer result.

When attempting an estimate of the *general average* rate of incrustation of large pipe-lines in the British Isles, he was inclined to adopt a somewhat higher figure than that suggested by Mr. Lloyd-Davies. It would seem to be rather misleading to adopt the Elan aqueduct figure as an average, seeing that in that case the water was rough-filtered before entering the pipes. Moorland drainage-areas must be expected to yield water of a somewhat peaty or acid nature, which was specially favourable for the growth of nodular incrustations; and many schemes involved the abstraction of water from such areas. In conformity therefore with the relation between the constants as found by Mr. Lloyd-Davies, he suggested the equation $e = 10 t^{0 \cdot 4}$ as being of general service for untreated moorland water; but this was to be regarded not as the average but as the minimum loss which might be expected. This equation gave the following losses (e) in discharging-capacity at the various ages (t) named.

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[1] Minutes of Proceedings Inst. C.E., vol. ccviii, p. 62.

[2] *Ibid.*, p. 25.

Mr. Barnes.

Age in Years.	Loss in Discharge.	Age in Years.	Loss in Discharge.
	Per Cent.		Per Cent.
1	10·0	10	25·1
2	13·2	15	29·5
3	15·5	20	33·1
4	17·4	25	36·2
5	19·0	30	39·0

Mr. Braine.

Mr. C. D. H. Braine observed that the estimate of the water available for filling the Mazoe reservoir appeared to be liberal. The maximum of 55·76 inches was recorded at Borrowdale in 1917–18 (Appendix A) and could scarcely be applied to the Mazoe catchment, for the maximum for Mazoe for the same year was only 41·36 inches (Appendix B). The average of 33·46 inches seemed high, considering that the gauge was really off the catchment, and that the average at Salisbury was only 31·86 inches, and at Cleveland 30·70 inches. The records at Volynia and Borrowdale extended over too short a period to be reliable. Appendix C gave a discharge of 61·96 per cent. for a rainfall of 0·81 inch, and a 35-per cent. discharge for a rainfall of only 0·42 inch. The information supplied must have been inaccurate. The conditions at Mazoe appeared to be very similar to those at the Hex river in the Transvaal. The latter had a catchment-area of 198 square miles and an approximate mean annual rainfall of 26 inches, ranging between 16 and 40 inches. The maximum discharge recorded was between the 15th and 18th December, 1914—a year of unusually heavy rainfall. During that time 6·47 inches fell, and the discharge was only 20·2 per cent. The average discharge for 8 years was 4·54 per cent., including the abnormal year 1914–15, when the total discharge was 13·94 per cent. Eliminating the year 1914–15, the average discharge was only 3·195 per cent., with a minimum of 1·69 per cent. It would be interesting to know how the Author calculated the normal flow. In the Union of South Africa the method of defining or fixing the normal flow was thus prescribed in Act No. 8 of 1912 :—

> The normal flow of any particular public stream or of any particular portion of a public stream and the surplus water thereof, shall be determined, in accordance with regulation, by the water court concerned.

The Author assumed an absorption of 13,500 acre-feet out of a

Mr. Braine. total capacity of 17,500 acre-feet. That amounted to the prodigious figure of 77 per cent. of the total storage. Had any such losses been recorded to justify that assumption? The section of the wall seemed on the heavy side. The formula[1] $T = RP/S$, allowing a stress of 20 tons per square foot, would give a base width of 24·116 feet, and the base would be 25·92 feet by the formula $t = r\,(1 - \sqrt{1 - 2p/\sigma})$, in which t denoted the thickness of the masonry at a horizontal section where the water-pressure was p, σ the maximum permissible compressive stress, and r the outer radius of the arch.[2] The description of the cementation process was interesting and useful. The reason for not using percussion drills for the deep holes was that the flushing and cleaning of the holes became tedious, and it was found that the diamond drills were more economical. The proportion of cement to water used for grouting ranged from 1 to 3 per cent. by weight. The cementation process had been used recently in connection with the foundations of the grain-elevator at Capetown.[3] The site was on made ground consisting of ashes, clay, sand, rubble and rubbish, through which the sea-water rushed so freely that pumping was impracticable. The cementation process proved entirely satisfactory. In a less perfect form cementation had been used near Homburg in 1864. The total leakage appeared to be about 3·20 cusecs, and was described as a small percentage of loss in the storage-efficiency of the dam. That, of course, was correct; but, when reduced to acres, it represented a considerable reduction of the irrigable area. Taking the duty as 150 acres, it amounted to a loss of 480 acres; and, as some crops only required irrigation during a portion of the year the loss was still greater. The pick-up weir referred to could not compensate for the leakage, as there were periods when no irrigation was required.

Mr. Hague. Mr. H. W. HAGUE observed that, since Mr. Ross's Paper was written, an overflow had been provided in the dam by means of a tunnel almost vertically over the outfall tunnel. There were now no Venturi meters on the inlet and discharge pipes. The valve at the lower end of the outlet-main near the distributing-basin was never used, as the joints would not stand the pressure. Mr. Ross left Hobart in 1918 and was succeeded by Mr. H. E. Bellamy, who, on the 23rd March, 1921, recommended the laying

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[1] "Concrete and Masonry Dam Construction in New South Wales." Minutes of Proceedings, Inst. C.E., vol. clxxviii, p. 2.

[2] Capt. A. ff. Garrett, R.E. "The Theory of Arched Masonry Dams." Professional Papers of the Corps of Royal Engineers (Fourth Series), vol. ii. Also in Tudsbery and Brightmore's "Waterworks Engineering," 3rd ed. London, 1905.

[3] "Engineering Abstracts," No. 17, p. 48.

Mr. Hague.

of a new 30-inch main. The reinforced-concrete supply main had not proved the success anticipated, chiefly owing to its being laid on made ground. The position was regarded so seriously that the Board of Engineers, in their Interim Report of the 10th October, 1922, recommended the relaying of a length of 13,065 feet at a cost of £5,000. The Hobart City Council was using reinforced-concrete pipes for storm-water and sewerage purposes, with complete success. The 4- and 6-inch pipes were now supplied with the collars woven on to the pipe, and that reduced the labour of collaring each pipe, which was usually done on the bank for the larger sizes. The dam and the valve-tower at Ridgeway had developed serious leaks, but these had been repaired, and both were now in good order. It had been found necessary to reline the valve-tower. From the figures given by Mr. Ross it appeared that before the construction of Ridgeway reservoir the storage-capacity was only sufficient for 27 days' supply, and with Ridgeway to 60 days' supply. Such figures were remarkable for Australia, and in the early part of 1920 it was necessary to impose the most drastic restrictions on the use of water. Consideration had been given to the question of increased storage, but owing to the failure to find a suitable site on the slopes of Mount Wellington, Mr. Bellamy and the Board of Engineers consulted had recommended the bringing-in of an entirely new supply of 6 million gallons per diem from the Humbolt river and the lakes of the National Park, 50 miles distant, at a cost of approximately £500,000.

Mr. Jennings.

Mr. A. C. JENNINGS observed that the discharge-figures given in Mr. Randall's Paper related to years in which the rainfall was considerably above the average. The rainfall during the season 1917–18 was the highest experienced in the past 30 years, for which records were available. The seasons 1918–19 and 1919–20 were also above normal. The reservoir had a total capacity of 17,500 acre-feet, and to fill it required a discharge of 7·60 per cent. from a normal rainfall of 33 inches, well distributed over the catchment-area. The following were the figures for the years subsequent to those given in the Paper :—

Season.	Rainfall.	Storm Discharge.	Normal Flow.	Total Discharge.
	Inches.	Per Cent.	Per Cent.	Per Cent.
1920–21	37·49	7·41	4·82	12·23
1921–22[1]	23·45	3·03	1·83	4·86

[1] A bad year.

Mr. Jennings. It would be evident that the dam was not likely to fill in all years, especially if only a portion of the normal flow was allowed to be impounded, which would possibly be the case, as irrigation development extended lower down the river. The figure of 16·20 per cent., stated in the Paper as being the probable discharge from that type of catchment-area, was too high and was not likely to be attained, except in years of rainfall considerably above normal. As Government Irrigation Engineer in Southern Rhodesia, he had visited the works frequently during their construction and also during the 3 years since their completion. The cementation of the foundation rock was undoubtedly a notable feature of the work, and had rendered possible the construction of a high concrete dam on a very poor foundation. The process, to the extent adopted at Mazoe, could not be applied to every foundation, particularly in the case of a gravity dam, where the risk from uplift due to a porous foundation would be very great. It appeared doubtful whether the very high pressures used for pumping in the grouting were really necessary; later works in which dam-foundations had been similarly treated, notably Lake Mentz dam[1] in the Cape Colony, had been completed with pressures not above 100 lbs. per square inch, although admittedly the rocks were of very different types. Where the fissures were filled with clay or earthy matter, there appeared to be a tendency for that to become consolidated, so that it was not displaced by the grouting, even under the high pressure used. Such an occurrence had been noticed when excavating in the vicinity of certain of the treated holes. The leaks noted by Mr. Randall did not appear to have improved to the extent he had anticipated. Unfortunately, the small gauging-weir below the dam had become damaged, and it had not been possible to maintain regular gaugings; but from careful observations and approximations the total discharge below the left-bank spillway was estimated to be not less than 4 cubic feet per second when the dam was full. The spillways of the dam did not appear to have received the consideration that such a high structure warranted, especially in view of the admittedly inferior nature of the rock in the flanks. In the right-bank spillway the horizontal concrete apron shown in Figs. 7, Plate 5, did not appear to have been placed on a solid foundation, and that, together with one of the spur walls, had been destroyed by flood in 1921. Subsequent floods, especially those during March, 1923, had scoured to an enormous extent the loose decomposed rock immediately below the toe of that spillway, and it was thought that very considerable protection work would be required there at an early date. The rods operating the valves on the up-stream face of the wall (Figs. 11, Plate 5) had had to be entirely renewed recently. The rods were round mild-steel bars 2 inches in diameter (not 2½ inches, as stated on the Figure). They were supported in iron brackets 10 feet apart between centres. Shortly after the works were taken over from the contractor it was found that certain of the gates could not be effectively operated, and when the water in the reservoir had fallen it was seen that the rods had become badly bent between the brackets. The primary cause appeared to have been seizure of the rods in the brackets. No linings were provided in the latter, and they soon became rusted. It was assumed that the attempt to operate the gates under such conditions caused more severe bending. New brackets fitted with gun-metal roller bearings had been fixed 5 feet apart between centres, and certain of the rods had been renewed. The work was completed some months ago, and up to the present had given satisfaction. It had been suggested that distortion of the arch due to temperature and other stresses was probably responsible for the initial seizure of the rods, but he did not accept that view. Experiments to determine the arch deflection of the dam were now being conducted. **Mr. Jennings.**

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[1] "Engineering Abstracts," No. 14, p. 67.

Mr. Mitchell. Mr. George Mitchell remarked that it would be interesting to know exactly why Mr. Ross's predecessor proposed masonry for the reservoir-dam, when it could presumably have been built much more cheaply of earth. With regard to the use of spun reinforced-concrete pipes, the Paper was most timely, as there were several large aqueducts projected at the present time. It was a truism that difficulties in constructing things generally increased in a far greater ratio than that of size. This applied especially to spun pipes. It appeared to be comparatively easy to get satisfactory pipes of the smaller diameters for low pressures, but for pipes of 3 or 4 feet diameter there did not seem to be sufficient experience yet to produce, at a reasonable cost, satisfactory pipes for medium pressures. As the Author indicated, the working-stress in the steel must be kept very low in large pipes. That meant two heavy spirals of close pitch, and it appeared to be very difficult to ensure that the spirals kept their proper place unless expensive means were adopted. In practice a good deal of welding was essential to make sure that deformation and displacement of the reinforcement did not take place during spinning. The question of the allowance for water-hammer was a difficult one. Was it safe to assume that in a large pipe a slight

Mr. Mitchell. temporary cracking under water-hammer pressure was permissible? Probably it might be better to spin on a ⅛-inch lining of bitumen, as had been done on the Continent recently. That would prevent the percolation of water in the concrete and so possibly overcome the trouble caused by the stretching of the steel under stress. There did not seem to be any exact experimental knowledge of the behaviour under stress of a spiral reinforcement which was all very slightly out of truth. In large pipes the resistance to external pressure from earth, etc., was an important factor in the design. Such pressure was very difficult to provide for, and nothing but extensive experiment on large pipes would give really satisfactory data. For considerable pressures it did not appear to be economical to use reinforced-concrete pipes unless the pipe practically became a solid steel welded cylinder, lined on one or both sides with cement mortar. The largest American makers of reinforced-concrete pipes stated that they were not economical for pressures exceeding about 70 lbs. per square inch. The design and construction of reinforced-concrete pipes was becoming so important for water- and sewerage-works, etc., that the Ministry of Health ought to have funds for enabling the National Physical Laboratory to carry out exhaustive tests on reinforced-concrete pipes. The perfecting of such pipes would no doubt be beneficial to the public in promoting the improvement of metal pipes and, especially, means to resist the corrosion of metal, which really was the key to the whole situation.

Mr. Olive Mr. W. T. OLIVE remarked on the extravagance of adopting the water scheme of Steenbras, in which a main about 39 miles in length had been laid at a cost of £800,000, whilst about 10 square miles of mountain land existed in the Peninsula, above the 1,000 feet contour, undeveloped as regards available rainfall for water-supply purposes. The Steenbras waterworks appeared to have cost £5 17*s.* 6*d.* per unit of the whole population, half of which was coloured. Ten years ago the consumption was officially stated to be 32 gallons per head, but only one-half of that appeared in the main outfall sewer. Careful measurements and observations were made continuously every ½ hour for 24 hours to arrive at that figure,[1] indicating abnormal waste, or misuse. Two years after the completion of the Steenbras scheme it was announced in the local press that the average daily consumption in January, 1920, was 5,038,000 gallons, but that in 1923 it had increased to 9 million gallons. With practically a stationary population, that required some expla-

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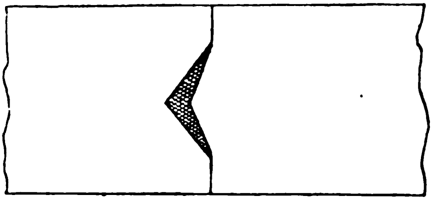
[1] Report to Town Council, Feb. 1913.

Mr. Olive. nation; for it could scarcely be contended, with conditions such as obtained in Capetown, that the actual consumption was 45 gallons per head. However, despite the assurance of a supply of 5 million gallons a day for 10 years, from a 600-million-gallon reservoir, the supply had been now, after the lapse of 2 years, cut off between 6 p.m. and 6 a.m., and engineers had been called in to advise. It was stated that during 1922 a diminution of 5 per cent. was observed in the main. He doubted the usefulness of any universal formula for the percentage loss of discharge, on account of the varied qualities of waters and the fact that a constant used fluctuated between 1 and 3.

Mr. Reynolds. Mr. J. F. J. REYNOLDS observed that his experience was based upon the laying of about 4 miles of 6- to 15-inch spun reinforced-concrete pipes. The type of joint shown in *Fig. 4* (p. 307) did not allow the pipe to centre itself, and had been modified as shown in *Fig. 2*, with satisfactory results. Although the spigot end was frequently chipped during transport, sufficient remained for centering purposes. The method of jointing recommended by the makers was first tried, namely, the pipes were half-jointed, clear of the trench, by caulking with a 1-to-1 mixture of cement and fine sand, as described in the Paper, and, after 24 hours for setting, were lowered into the trench, the jointing being then completed with plastic and caulking. That method proving slow, the following was tried:—Two holes about 1 inch in diameter and 4 inches apart were left in the centre of the collars. The pipes, each with a loose collar threaded on, were lowered and bedded in the trench, the plastic joint was made, the pipe was driven home, and the collar was centered over the joint. The ends of the collars were then caulked with yarn with the grout-holes on top, and the annular space was filled with strong, neat, slow-setting cement grout, just sufficiently fluid to run freely into one hole through a large funnel. When the grout rose in the second hole, it was "pumped" gently to exclude air and surplus water, any settlement

Fig. 2.



Mr. Reynolds. being made up as necessary. Although, owing to the yarn ends, these joints were sometimes 2 inches shorter than the collar, they were perfectly satisfactory and showed less tendency to sweat under test than did the caulked joints. That method was quicker and cheaper, and required less excavation at joint-holes than for the caulked joints. For pressures up to 10 lbs. per square inch, a square step joint, similar to *Fig. 5* (p. 307), but without plastic material or mortar, covered on the outside by a strip of adhesive tape and completed by grouting, as just described, should be a satisfactory joint for many purposes. Mr. Ross had not drawn attention to the tendency of this class of pipe to sweat freely under test, even with a head of only 4 or 5 feet, the quantity depending upon the age of the pipes and the condition of the atmosphere when they were laid. That appeared alarming when first seen; but, after a period, depending upon the same conditions, the pipes and joints would dry and remain watertight. The most likely point of failure was at the longitudinal joint of the mould; if that were not tight, the finer particles of the concrete might be thrown out during spinning, leaving a porous seam. Mr. Ross recommended the placing of the reinforcement in the outer third of the thickness. While that was desirable for theoretical reasons, for practical purposes it was better to wind the reinforcing wire so that the cage would occupy the middle third, taking care that the joints in the wire were at the ends of the pipe, where they would be protected by the collar, if too near the skin. The position of the reinforcement, not being registered in the thickness of the pipe, was entirely dependent upon centrifugal force.

Mr. Temple. Mr. F. C. TEMPLE observed that Mr. Ross's Paper was of special interest to engineers in India at the present time, because the use of reinforced-concrete pipes made by the Hume process had only recently been introduced there, and was the subject of much difference of opinion. Experience had shown that there were considerable difficulties to be overcome. The first pipes made were matured in the air, covered with bags kept wet by sprays. That was satisfactory in the rains; but pipes so matured in the dry weather failed under test. Other pipes intended for very much lower pressures, which had been rolled into a borrow-pit full of water, were found to stand much higher tests, and maturing under water had been adopted throughout. Fully-matured pipes, however, behaved in a remarkable way. If they were kept in the air for any appreciable length of time in the dry weather, they were liable to sweat very heavily when first put under pressure. If kept under pressure for 3 weeks to a month, they dried completely, unless a **Mr. Temple.** definite crack occurred. That was probably due to the extraordinary differences in the humidity of the atmosphere, which ranged from 100 to 10 per cent. for 8 a.m. readings, and had been known to go as low as 5 per cent. at 4 p.m. Certain experiences had been similar to those at Hobart. Bad foundations had given great trouble until it was realized that the pipes must be packed and bedded with extreme care throughout their length. For that reason, where an 18-inch pressure pipe had to cross a stream on pillars, rails were laid on each side of the pipe, and wedges were driven in between the rail and the collar, and subsequently concrete was packed between the rail and the pipe throughout its length. A pipe laid in a wet trench and exposed to hot sun was specially vulnerable. Distorting stresses appeared to be set up, and many pipes laid in such situations had developed cracks. That has been overcome by covering the pipe with earth, immediately after laying, to a depth of about 6 inches. If leakage occurred when the pipe was put under test, it would show at once through that small layer of earth. Variations of temperature tended to crack the joints. The variations were frequently 30° F. between the day and the night temperature, and extended from about 45° F. minimum in the cold weather to 124° F. maximum in the hot weather. But the variations in the total length of a pipe did not appear to depend so much on the variations of temperature as in the variations of humidity and saturation of the concrete. For though the concrete was extremely dense and took up a very small percentage of water, that quantity appeared to be sufficient to make an appreciable difference in the total length of a long pipe-line. Two pumping-lines had recently been put into operation. One was 12 inches in diameter and some 6,000 feet long, and the other 18 inches in diameter and about 8,500 feet long. Both were fed from the same pumps, starting with a pressure of 80 lbs. per square inch, and ending with a pressure of 30 lbs. The 18-inch pipe delivered into a service-reservoir, which enabled the pressure to be taken off the pipe when repairs were required. The 12-inch pipe delivered straight into the mains, and various methods had to be devised for repairing under pressure. For instance, a blown-out joint had been caulked with lead wool sufficiently to make it possible to build a block of concrete around the joint, a ½-inch drain-pipe left through the concrete for the leakage not stopped by the lead wool being subsequently plugged when the concrete had set. Pin holes and hair cracks had been stopped by applying a small quantity of plaster of Paris and immediately binding on a

Mr. Temple. split collar and caulking it with cement. A pipe which had broken its back owing to settlement in the trench was repaired by means of a piece of sheet india-rubber, so arranged as to lead the water leaking from the crack to a drain-pipe while a cement block had time to set around the pipe. It appeared that the engineer laying concrete pipes in the dry parts of India must be an optimist full of faith. For a pipe-line might be streaming wet when first put under pressure, looking as if it could never stand, and yet be quite dry, and, as far as could be known, secure, in 3 weeks' time.

Mr. Thorpe. Mr. W. H. THORPE observed that, though strict calculation of arched dams was not very convincing when checked against successful practice, yet a useful comparison of examples was possible if horizontal arch-rings were considered as subject to simple compression due to hydrostatic pressure. For that purpose, varying a common expression, thickness might be said to equal $\frac{\text{radius} \times \text{depth}}{C}$, C being a quantity which for the same material should be of constant value. Modifications of stress due to fixity of arch-ring ends, and fixity at the base, though existent, were in this candidly ignored, but the quantity C, as deduced from examples, became a fair gauge of strength values for similar materials. On that basis:

For the Hobart dam . . . . $C = 705$
" " Mazoe " . . . . $C = 540$
" " Capetown " . . . . $C = 366$

For twenty-seven cases of concrete or rubble arched dams (not reinforced), including those described in the three Papers, and some others of recent date, the constant had values ranging from 359 to 1,923, with a mean value for the twenty-seven cases of 634. That indicated a difference in practice not to be accounted for by differences in material, and could hardly be considered satisfactory. If sound practice were supposed to be represented by a mean value of 600 for C, it would appear that no reduction of section would result from adopting the arched form for any radius exceeding 400 feet, for which radius "thickness" would be two-thirds of the height. It was questionable, however, if any sensible increment of strength resulted from arching a dam approaching gravity section, though it might be of advantage as a safeguard against localized weakness.

Referring to the manufacture of reinforced-concrete pipes by the centrifugal process as described by Mr. Ross, he thought the effect of whirling in consolidating the concrete was evidenced by the great weight of the material per cubic foot, giving the less occasion for surprise that so high an aggregate resistance should be developed. He estimated that the consolidating pressure for the 4-inch pipes at the maximum number of revolutions stated would be equivalent to a 2-foot head of concrete, and for 6-foot pipes twice as much, and he thought it would be well to know whether the difference in setting-time referred to had any connection with the number of revolutions per minute, that was to say, whether it was affected by differing pressures, and perhaps accounted for the results noted. Mr. Thorpe.

Mr. Lloyd-Davies. Mr. **Lloyd-Davies**, replying to the Discussion and Correspondence, thanked Mr. Barnes, the author of the original pipe-deterioration formula, for his kind reception of the generalized formula suggested in the Paper. A *general average* formula should only be adopted in the absence of any data to serve as a guide to the most probable values of the constants p and q. It was probable that the formula for the Elan aqueduct, though approximately a mean value of the various formulas shown by curves in *Fig. 12*, was unsuitable for the particular case of untreated British moorland waters, and that the modification suggested by Mr. Barnes would be more appropriate to that case.

In regard to Mr. Gourley's remarks, it should be stated that the application of well-known pipe-discharge formulas to the case of the Steenbras pipe-line was subject to whatever error might be incidental to the use of a nominal diameter. But the deterioration formula adopted was based on the successive meter readings, not upon a calculated initial discharge. The empirical formula adopted for the average rate of deterioration of a long pipe-line should be consistent with a wide range of variation among the different pipes which contributed to the average result. That derived for the Coolgardie main, referred to by Mr. Binnie, did not, and need not, fit all sections of that main taken separately; and it was possible that certain sections might have so exceptional a history as to be unadapted to any general formula for pipe-deterioration. Moreover, it could not be expected that the same formulas would apply equally well to steel mains and to cast-iron mains. The data given by Mr. Binnie were not essential to the derivation of the formula by which a relationship was found between the constants p and q in the typical formula $e = pt^q$. It was a fact, however, that the mean between the extremes mentioned by him, namely, 28 per cent. deterioration in 5 years, was nearly in accord with the curve $e = 18{\cdot}5\, t^{0{\cdot}30}$ (*Fig. 10*), which gave 30 per cent. deterioration. Curves might be plotted separately to fit the extreme data, namely, Mr Lloyd-Davies. 12 per cent. and 44 per cent. respectively, in 5 years. These would have the formulas (approximately) :—

$$(a)\ e = 5{\cdot}2\, q^{0{\cdot}53}$$
$$(b)\ e = 33\ \ q^{0{\cdot}23}.$$

If carried on for a further period of 7 years, making 12 years in all, the respective deteriorations of these two categories of pipes would be (*a*) 19 per cent., (*b*) 55 per cent., according to the above curves when plotted to the scales of *Fig. 10* given in the Paper. A separate study of the history of different siphons on the Coolgardie main would be necessary to establish or refute a correspondence with the above, or similarly derived, formulas.

⁂ Mr. Randall's reply (if any) to the Correspondence will be printed in a subsequent volume.—SEC. INST. C.E.

10 April, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The PRESIDENT announced with regret that The Institution had lost by death one of its Honorary Members, Sir James Dewar. The Council had passed the following resolution, with which, no doubt, the members would concur :—

" That the Council record the deep regret with which they have learned of the death of Sir James Dewar, M.A., LL.D., F.R.S., who has been an Honorary Member for 22 years and whose distinguished services to science have been valued and appreciated most highly by the members of The Institution."

It was resolved—That Messrs. A. H. Case, E. R. Dolby, W. Fairley, J. M. Kennedy, S. C. Lewis, A. J. Martin, A. H. Preece, and R. E. Tickell be appointed to act as Scrutineers, in accordance with the By-Laws, of the ballot for the election of the Council for the year 1923–24.

The Council reported that they had recently transferred to the class of

Members.

FRANK STEWART EASTON, B.Sc. (*Glas.*).
LUIZ DE MORAES GOMEZ FERREIRA.
JOHN HERBERT HAISTE.
ARTHUR FREEMAN HOLDEN.
Professor CHARLES EDWARD INGLIS, M.A. (*Cantab.*).
FRANK WILLIAM IRELAND.
CHARLES WILLIAM JENKINS.
JOHN WILLIAM MCLAREN, B.Sc. (*Glas.*).
GEORGE VINCENT MAXTED.
PERCY EDWARD SHEPHERD, O.B.E.
WILLIAM HERBERT SHIELDS, B.Sc. (*Glas.*).
ARCHIBALD HUGH SMYTH.
EDWARD HUME TOWNSHEND, B.A.I. (*Dubl.*)
JOAH HAIGH WALKER.

And had admitted as

Students.

STANLEY CARLIN BARRIE.
FRANK BRAIN BELTON.
CHARLES CHAMBERS, B.Sc. (*Edin.*).
LAXMAN VASUDEV CHHATRE, B.E. (*Bombay*).
BERTRAM BENJAMIN COHEN.
REGINALD WILLIAM HULME COUZENS, B.A. (*Cantab.*).
HENRY KINLOCH COX.
CHARLES GORDON DERBYSHIRE, B.Sc. (Eng.) (*Lond.*).
JOHN KENNETH FISHER, B.Sc. (Eng.) (*Lond.*).
EDMUND AUSTIN GARDINER.
JOHN GAGE GARDINER.
WILLIAM RONALD GILCRIEST.

Students—continued.

Richard Cooper Gowthorpe, B.Sc. (*Manchester*).
William Evelyn de Courcy Hamilton.
David Kenneth Hopkins.
Narendra Nath Kakati, B.Sc. (*Bristol*).
Cyril Lambert Kendall.
Sidney Haighton Medcalf.
Donald Brockholes Harvey Moore.
Cedric Stewart Toller Paul.
Herbert John Pearce.
John Penman.
Guglielmo Giacomo Edoardo Scribante.
Robert William Stanton, B.Sc. (Eng.) (*Lond.*).
George Ballam Hilton Sutherland.
Cecil William Morgan Williams.

The Scrutineers reported that the following candidates had been duly elected as

Members.

Charles Frederick Farrington.
William Robert Gerald Whiting, M.B.E., M.A. (*Cantab.*).

Associate Members.

Percy Neville Humphry Baker.
Horace Ernest Comben, B.Sc. (Eng.) (*Lond.*).
Henry Graham McDowell.
William Paterson.
Arthur Howard Prince.
George Frederick Renton, Stud. Inst. C.E..
Sidney Rouse.
Edward Peel Stevenson, B.Sc. (*McGill*).
Harald Dettloff Thomson, B.Sc. (*Glas.*).
Joseph Wright, Stud. Inst. C.E.

(*Paper No. 4410.*)

"The King George V Dock, London."

By Asa Binns, M. Inst. C.E.

Introductory.

The Port Authority came into existence on the 31st March, 1909, having been constituted under the Port of London Act, 1908. A programme of new works and improvements, designed by Mr. Frederick Palmer, C.I.E., M. Inst. C.E., the first Chief Engineer, was provisionally adopted by the Authority towards the end of 1910 to meet growing requirements as nearly as could then be foreseen, and the largest and most important of these new works approved for immediate construction is described herein. The detail designs were not taken in hand until the autumn of 1911, the delay being occasioned principally through the necessity of co-ordinating the divergent opinions concerning the dimensions of the passages and even of the dock itself. In the meantime a careful survey was made, including cross sections at intervals of 100 feet over the whole site, and a series of twenty-eight borings and two trial holes were sunk. A contract for the work was placed with Messrs. S. Pearson and Son, Ltd., in August, 1912, one of the terms being that the work should be completed in 4 years.

The works were brought nearly to a standstill by the war, and it was not until the autumn of 1918 that facilities for obtaining an adequate supply of labour and materials were granted by the Government. In order to meet urgent national demands, the contract was then terminated by arrangement, and the Port Authority completed the works by direct administration.

The site lies partly within the boroughs of Woolwich, East Ham, and West Ham, and has the advantages of deep-water access from the river, good railway connections, and it is within easy cartage distance from the City. The general level over the site was several feet below high water. The borings and excavations showed Thames alluvium overlying the chalk. At the western end of the site Thanet sand is found underlying the valley drift, but over the greater portion of the site it has been eroded. Neither the London clay nor the Woolwich beds are present. The alluvium consists of clay, lignite or peat, and ballast. In the peat and clay detrital wood and water-logged trunks of trees were common. Fresh-water shells, hazel-nuts, reeds, etc., were also found. Several silted-up creeks cutting into the peat were encountered in the course of the excavations, the largest of these being the old Ham creek which in the seventeenth century was navigable. On the whole the geological conditions prevailing over the site are exceptionally good. The heavy works are founded generally on the ballast or chalk; the soft alluvium made easy getting for the general excavations, and the ballast, being of excellent quality, was used for practically the whole of the concrete work. A considerable number of houses on the site had to be demolished, and the Port Authority carried out a re-housing scheme at Prince Regent's Lane, where 204 new houses were built on garden-city lines with twenty-two houses to the acre.

Design and Method of Construction.

General Description.—The new works (Figs. 1 and 2, Plate 7) comprise a wet dock with an area of approximately 64 acres, a depth of 38 feet of water, and a total length of quay of just over 2 miles, giving berthing accommodation for fourteen large vessels. The approach from the river is gained by an entrance lock 800 feet long by 100 feet wide, with a depth of water over the sills of 45 feet at Trinity high water, or 41 feet 8 inches at H.W.O.N.T. The dry dock is 750 feet long and 100 feet wide with a depth of 35 feet of water over the keel-blocks. The wet dock is connected to the existing Royal Albert dock by a passage 100 feet wide with a depth of 34 feet (Figs. 2, Plate 7).

Wet Dock.—A feature in the design of the wet dock is the provision of reinforced-concrete jetties parallel with the south quay-wall, with a barge passage between the jetties and the quay (Figs. 1, Plate 7). Cranes on the jetties work the ships' cargo either into barges or to the quay and transit-shed as may be required. The dock-bottom slopes upward at a 1-in-5 gradient from the outer face of the jetty to the face of the quay-wall, thus reducing the height of the wall by 11 feet when compared with the north quay-wall where the jetties are absent. Detailed cross-sections of the two walls are given in Figs. 3, Plate 7. In each case the wall has a projecting toe and rests on a sloped foundation. The face is vertical. The concrete is 8-to-1 quality, excepting a 6-inch face of 4-to-1 mixture. The calculated maximum pressure on the foundation at the toe is 5 tons per square foot. The excavation down to the ballast was carried out by means of two Lübecker land-dredgers. Behind the walls 1-to-1 slopes were formed. Below the ballast-level the excavations for the walls were carried out in timbered trenches. The ballast over the dock-bottom was removed by means of grabs and was utilized for concrete or for filling behind walls.

Entrance Lock.—The entrance lock and river wing-walls were kept well back from the river-bank to enable the whole of the work to be carried out in the dry; thus an artificial river-dam was not needed. Another object of this set-back was to form a bell-mouth for sheltering vessels when entering the lock. The plan and sections, Figs. 4, Plate 7, show the general design. There are three sets of gates, which are interchangeable. The levelling-culverts are short and unlined, those at the outer gates being disposed to sweep the apron, which is prolonged to 150 feet beyond the heel posts. The outer extremity of the apron is turned down 10 feet into the ballast to prevent underscour. The invert and walls are constructed of mass concrete with granite sills, quoins, coping, etc. Below the springing of the invert the concrete is of 6-to-1 quality, and above that level it is of 8-to-1 quality, excepting the 6-inch facing of 4-to-1 quality as already described in the case of the wet-dock walls. Dam grooves are formed to enable each gate-platform to be pumped out for heavy repairs if necessary. At the inner end of the lock is a stop for a floating caisson, which will increase the effective length of the lock to 910 feet. The dressing of the watertight faces of the granite gate-sills and quoins, caisson-stops, and penstocks was carried out with the utmost care by stretching fine wires and by the use of long steel straight-edges. Six subways pass under the lock with shafts on each side, formed in the mass concrete of the invert and side walls, for the accommodation of gas-, water-, and sewage-pipes, and electric cables. The shafts and subways are generally 6 feet in diameter and are lined with cast-iron segments.

The diverted mains comprise a 12-inch gas-main, a 20-inch steel water-main, two 14-inch and one 24-inch sewage-pumping mains. These are carried to and from the lockside on reinforced-concrete rafts.

The excavation, with suitable slopes behind the walls down to ballast-level, was carried out by grabs and a steam navvy; and timbered trenches for the side walls were excavated in the ballast and chalk by means of grabs. The walls were then carried up to within 6 feet of the coping-level. For this purpose two gantries 132 feet long, spanning the lock, were mounted on pedestals supported on heavy bogies running on rails laid on the ballast. Each gantry carried two 5-ton cranes which handled the concrete skips for building the walls. For dealing with the invert of the lock the gantries were carried on bogies running on tracks laid on the side walls at the level mentioned above; the pedestals were taken away, and the cranes removed the excavation by grabbing and placed the concrete in position. The topping up of the walls, including the formation of the pipe-subway, was completed at a later date with a 5-ton locomotive crane on track laid on the filling behind the walls. The river wing-walls at the entrance of the lock were constructed in timbered trenches, the foundation-level rising in four steps of 4 feet each from −40 to suit the 1-in-5 dredged slope from the entrance channel. On the south side of the lock the wall is continued to afford a berth 400 feet long with a depth of 31 feet 6 inches at Trinity high water. The excavation of deep trenches so close to the river was attended with considerable risk, and the work was carried out in short lengths. Occasionally the seepage gained on the pumping about the time of high water, but no serious blow took place, and the work was satisfactorily completed. The continuation of the north wing-wall was carried out in concrete sheet piling for a length of 164 feet to provide a barge berth. The sheet piling is tied back by means of a Walker-Weston reinforced-concrete raft forming the quay, which is anchored on its landward boundary by being turned down well below the line of the natural slope of the ground.

Dry Dock.—The plan and sections, Figs. 5, Plate 7, show the important features of the design. The entrance, on each side of which is a short filling-culvert, is closed with a floating caisson. The drainage-culvert to the pump-well is short, of large section, and with ample grating area. There are two timber slides with steps on each side of the dry dock. The side walls are carried through the head to facilitate future lengthening of the dock if required. A 6-foot diameter subway passes under the entrance with vertical shafts on each side similar to those already described at the entrance lock. The concrete is generally of 8-to-1 quality with a 4-to-1 facing; the sills, quoins, altars, shore stones, and coping are of granite. To relieve the pressure under the floor and behind the walls, vents are placed at 50-foot intervals just above the bilge altars. The invert was carried out in longitudinal strips instead of the usual transverse strips. The keel-blocks are of cast iron, carrying a 9-inch greenheart top block with a 3-inch soft-wood capping, and they are machined on their sliding faces. The total height inclusive of capping is 4 feet, the width is 4 feet, and the thickness is 15 inches, and they are spaced at 4-foot centres. A 25-ton electric travelling crane runs the whole length of the dry dock on the northern side, the outer track being carried on reinforced-concrete piles and beams. The radius of the crane is 80 feet, and it will thus plumb the centre of the dock. A 5-ton steam crane, 37 feet 6 inches radius, commands both sides of the dock. Compressed air is provided for the working of pneumatic tools for ship repairs.

Passage to the Royal Albert Dock.—The feature of special interest in carrying out this portion of the work was the method of removing the old dock-wall and connecting up the new bell-mouth walls of the passage. A single-skin pile dam 400 feet long was driven in the Albert dock clear of the toe of the old dock-wall. Part of the dam was made of steel sheeting, but more than half was of 12-inch reinforced-concrete piles. The dam was strutted from the old wall, and, as the demolition of the wall proceeded, the water was pumped down as required, and new struts were inserted, carrying the pressure to the timbering retaining the earth at the back of the wall. The concrete wall was broken up by small charges of gelignite and was removed, and the new wing-walls were connected up to the old dock walls without any untoward incident. A segmental concrete dam was built at the southern end of the passage abutting on the caisson-stops as a safeguard against the premature flooding of the works from the Royal Albert dock, due to the removal of the old wall. The emergency, however, did not arise, as the earth bank proved quite stable until the new dock was filled with water, when the bank was removed by dredging.

Jetties in Dock.—The seven jetties in the wet dock are constructed of reinforced concrete on the Considère system (Fig. 6, Plate 8). The piles are octagonal in section and measure 18½ inches across the flats. They were driven in the dry by means of a special pile-driving plant mounted on a bogie with a cross traverse to take the three lines of piles, the pile leaders being swivelled to give the required batter in either direction. The monkey weighed 83 cwt., and the average drop was about 6 feet. An average of five piles per day was driven under ordinary circumstances with a minimum penetration of 10 feet. The only difficulty in driving arose on the western jetty where Thanet sand was encountered, and here a water-jet had to be used. The lower back braces were pre-moulded with an eyelet which slipped over the pile, the surface being roughed at the point of articulation and the annular space between the pile and brace being grouted up. The deck of the jetty is of heavy section with special beams for carrying the crane-rails. The jetties are protected by timber fendering and copings; and cast-iron bollards are provided on both sides. A timber gangway which can be lifted by the cranes connects each jetty with the quay.

Jetties at River Entrance.—The entrance jetties are constructed generally in pitch pine with creosoted fir decking and joists, and oak sheathing (Fig. 7, Plate 8). A dozen or more large ships may be passed through the entrance in one tide, and, however skilfully handled, they are bound to fall heavily alongside the jetties from time to time: hence the use of an elastic material. The piles are 65 to 70 feet long, built up of whole pitch-pine timbers bolted together giving a minimum section 28 inches by 14 inches. They are shod with special cast-iron shoes weighing 190 lbs. each. For the two jetties 257 piles were driven, using a McKiernan-Terry pile-hammer and the special pile-frame with swivelled leaders already referred to, the entire plant being carried on a floating pontoon. The pile hammer weighed 6 tons and could deliver 100 blows per minute. Two or three piles per tide were driven under favourable conditions with a penetration up to 25 feet in ballast. No pile rings were used, the timbers being shaped at the top to fit the anvil-block, and occasionally the heads of the piles were set on fire by the heavy driving. The jetties are equipped with hydraulic capstans and the necessary fairleads and bollards. Landing-steps are provided at the back of the up-stream jetty.

Transit-Sheds.—On the south side of the dock are seven single-storey transit-sheds, each 528 feet long and 120 feet wide, in two spans of 60 feet (Figs. 1, Plate 7). They are carried on reinforced-concrete pile foundations and have steel frames covered with corrugated iron. Continuous skylights are fitted on the northern slopes. The floors are of concrete, and thirty sets of sliding doors giving openings 10 feet square are distributed round the sides and ends. The quay, which is 25 feet wide, is of reinforced concrete on the Walker-Weston system, and it is provided with two lines of railway-track with the necessary crossovers. For dealing with road and rail deliveries, the ends and south side of each shed are provided with platforms 10 feet wide paved with Victoria stone and sheltered by canopy roofs. On the south side are three lines of railway-track with the necessary crossovers. The cart areas at the ends of the sheds are 76 feet wide between platforms and are paved with granite sets on concrete. A block of latrines is provided in each cart area. A macadam road, 30 feet wide between curbs, runs south of the railway-lines, and additional loading-space for road vehicles is provided by means of reinforced-concrete paved areas connecting the road to the railway-platform. Each shed is provided with a customs lock-up, fire appliances, electric lighting, piling-machines, weighing-machines, and other gear and tackle. Three double-storey reinforced-concrete warehouses were built on the north side of the dock. The middle one is 1,150 feet long, and the end ones are 1,100 feet long. They are 120 feet wide on the ground floor, and the upper floor is 110 feet wide with a 10-foot veranda on the quayside. The clear height of the ground floor, measured to the underside of the main beams, is 15 feet, and of the upper floor to the roof ties, 20 feet. The three sheds are carried on reinforced-concrete pile foundations. The piles are 14 inches by 14 inches in section and range in length from 30 feet to 40 feet. Altogether 2,150 piles were driven, including those for the reinforced-concrete quay, which is 40 feet wide and carries two lines of railway-track and a 13-foot 6-inch gauge crane-track. The floors are designed for a safe load of 3 cwt. per square foot. On the ground floor the columns are spaced longitudinally 25 feet apart between centres and on the upper floor 50 feet. The panel walls are constructed of red Leicester brickwork, and the steel roof-trusses are of the saw-tooth pattern with continuous north lights. The sheds are divided by fireproof walls into four compartments on each floor, and hatchways are provided between the ground and first floors. The upper floor of each shed is equipped with eight 1-ton underslung electric travelling cranes, together commanding about 70 per cent. of the floor space. Two 1-ton electric bracket wall cranes are provided on each end of each shed for van deliveries from cart areas. A special feature of the sheds is the central cart area 100 feet wide, over which the first floor is carried on reinforced-concrete trussed beams supported by a central row of columns. Platforms 10 feet wide serve the cart areas and the railways on the north side, where there are three tracks with the necessary crossovers.

Dry Dock Pumping-Station.—The dry dock pumping-station floor is 27 feet below the coping-level of the dock, and is 88 feet long by 31 feet wide. The walls and floor are constructed of 6-to-1 concrete, and the walls above the glazed brick dado are lined with gault brickwork in cement mortar. The main sump is 15 feet wide with a floor 10 feet below the dry-dock floor. The main pumps, installed in duplicate, are designed to empty the dry dock in 3 hours when working together, which necessitates an average output per pump of 9,746 cubic feet per minute. Each pump has two impellers, a 48-inch diameter suction, and a 54-inch discharge. In each discharge-pipe is fitted a penstock valve worked by hydraulic pressure as well as a non-return valve. The main pump motors, of the two-phase, synchronous type with starting-motor and exciter, are directly coupled to the pumps. They run at 375 revolutions per minute and develop 820 B.HP. with a pressure of 6,000–6,600 volts, and a normal frequency of 50. The two priming-pumps, 15 B.HP. each, can charge either main pump in 10 minutes. Two drainage pumps, with 10-inch suction and 12-inch delivery pipes, can discharge 250 cubic feet per minute per pump at a speed of 930 revolutions per minute. They are driven by direct-coupled 40-B.HP. motors at 460 volts, continuous current.

The station is equipped with an overhead travelling crane and water-level indicators for the wet and dry docks. An enclosed staircase gives access to the station from the quay, and the switchgear is fixed on a gallery. Fans are provided for ventilation, and a small sump in the floor collects internal leakage and condensation.

Lock-Gates.—The segmental lock-gates are of the double-skin type with steel framing of continuous horizontal ribs and intercostal vertical diaphragms. Six air-chambers are formed in the lower part of each leaf, as shown in Fig. 8, Plate 8, and the upper chambers are flooded through scupper pipes. The gates are sheathed with timber on both sides and provided with rollers. They are operated by direct-acting hydraulic rams and have three hydraulic levelling-sluices in each leaf. The three pairs of gates are interchangeable, and each leaf, complete with sheathing and gangway, weighs about 350 tons. The gates are constructed generally of mild-steel sections and plates. The horizontal framing of each leaf consists of seventeen ribs, five of which form plated decks. Vertically each leaf is subdivided into compartments by five diaphragms, three of which form watertight bulkheads. The skin-plating, in strakes of about 6 feet, runs to lengths of over 20 feet. The horizontal joints are lapped, but the vertical ones are butted and furnished with double cover-plates. The thickness of the skin-plating ranges from ⅜ inch to ⅞ inch.

The heel-post, mitre-post, and clapping-sill are of greenheart. The heel-post is made in two pieces, each the full length, closely fitted and bolted to the steel core with galvanized countersunk wrought-iron bolts, and the holes are plugged with dowels of greenheart. The mitre-post is also in two pieces, each the full length, bolted together and to the steelwork. The clapping sill is formed in two lengths which are butt jointed. The sluice-trunks have clear openings 3 feet 5 inches by 2 feet 11 inches. The cast-iron doors and cast-steel guide-frames are fitted with gun-metal strips at the meeting faces. The hydraulic operating-cylinders are 7 inches in diameter and 3 feet 3¼ inches stroke, mounted on the back of the gate above high-water level.

The cast-steel heel-post pivot and socket castings are provided with forged-steel bearing-blocks, and are bedded on sheet lead, recessed into the granite pivot-stones and secured with galvanized lewis bolts. The cast-steel anchor-blocks are bedded on sheet lead in recesses in the granite quoin-stones and secured by galvanized lewis bolts. Each anchor-block is tied back into the masonry by three 5¼-inch mild-steel tie rods, each secured to the anchor-block by a gib and cotter, and fitted with a cast-iron anchor-plate and thrust-plate. The rods range in length from 17 to 24 feet and are passed through 9-inch sheathing-pipes built in the concrete. In fitting the anchor-plates and thrust-plates, with a view to avoid excessive elongation under stress, the rods were put under initial tension by means of long spanners before tightening up the cotters to the anchor-blocks. The forged-steel anchor-straps are bored true to the radius of the pintles.

The cast steel roller-paths are 1 foot 9 inches wide on the tread and machined top and bottom. The segments are machined at the ends and joined by cast-steel sole-plates with galvanized tee-headed bolts. The paths are bedded on the sunk granite with red lead and are secured by galvanized wrought-iron lewis bolts. Each cast-iron roller-carriage has a 5-inch diameter forged-steel pivot pin housed in the steel-plate cheeks of the roller-box. The bevelled cast-steel rollers are 3 feet in mean diameter with 7-inch diameter forged-steel axles and phosphor-bronze bearings. 1¼-inch steel-plate scrapers are fixed to give a half-inch clearance over the roller-path. The forged-steel spear posts are 6 inches in diameter, a short knuckle-jointed bottom length bearing on the roller-carriage. The top of the spear post is secured by a volute spring designed to give, through a movement of 3 inches, a pressure on the spear post approximating to 4 tons.

Each gate-operating machine has a cast-iron cylinder lined with gun-metal, the diameter being 18 inches and the stroke 13 feet 3 inches. The cast-iron piston-body is fitted with a junk ring for hemp packing. The mild-steel piston-rods are 7 inches in diameter, fitted into cast-steel crossheads working between mild-steel guides. The connecting-rods are built up of steel sections with steel gymbal attachments to the crosshead and "crocodile" lever, which latter is also of steel sections pivoted over the heel post in eye-plates secured to the anchor-block. The outer end of the opening lever is fitted with a cast-steel nose, 4½ inches in diameter, bearing in a steel plate secured to the top deck of the gate. A hydraulic ejector is fixed in each leaf, arranged with its connections so as to pump from each of the six air-chambers.

All three sets of gates were erected in the dry. The heel-post and mitre-post sections, and the sill girder, weighing from 28 to 35 tons, were delivered built up complete. A set of sheer legs 100 feet long of ample power and a 15-ton Scotch derrick were used for erection. On completion of the erection the whole of the buoyancy-chambers were tested under internal pressure. The final dressing of heel-posts, mitre-posts, and clapping-sills was carried out with great care, and each set of gates proved to be practically dry and watertight under a 20-foot head.

Floating Caisson.—The ship-type caisson is reversible, and is divided by two watertight decks and two bulkheads into five chambers (Figs. 9, Plate 8). Although the caisson is intended for regular use at the dry-dock entrance, it can also be used at the passage to the Royal Albert dock. To permit reversibility, the upper portion above the air-chamber is divided into two end compartments and a middle compartment, the capacity of the latter being equal to the combined capacities of the two former, and their connections to the outer water being on opposite sides of the caisson. A scuttle tank can be flooded through a 9-inch valve for sinking the caisson, and it is emptied by means of a hydraulic ejector. Anchor-stirrups fixed in the walls and engaging cast-steel noses on the caisson are provided as safeguards against accidental flotation. The caisson was designed so that its calculated centre of gravity was 22·68 inches below its centre of buoyancy at normal flotation with all tanks empty and with the flotation line 1 inch below the top of the air-chamber. Permissible stresses of 6½ tons per square inch in tension or compression were adopted in the design. The total weight of the caisson and its equipment, exclusive of permanent ballast is 608 tons. The weight of permanent ballast is an additional 357 tons. The skin-plating ranges in thickness from ⅜ inch to ⅝ inch. The angle frames, at 1-foot 11-inch centres, are of scantling ranging from 3½ inches by 3 inches by ⅜ inch at the top to 6 inches by 3½ inches by ½ inch at the bottom. The upper and lower decks forming the air-chamber are 7/16 and ½ inch thick respectively. Two double-faced hydraulically-operated sluice-valves with clear openings through the caisson 3 feet 6 inches square are provided for levelling. The caisson was built on the bottom of the dock before the water was admitted, a Scotch derrick on the quay-wall handling all the sections. On completion, and before the bitumastic coating was applied, the various compartments were separately tested by water-pressure. When the dock was filled, the caisson floated clear of its bearing-timbers and was towed into position at the dry dock.

Swing Bridge Over Passage.—The steel lattice-girder swing bridge over the passage is of the end lift type with fixed centre pivot, carrying a roadway 20 feet 8 inches wide between curbs and overhanging footways 5 feet wide on each side (Figs. 10, Plate 8). The total length of the bridge is 200 feet, and it spans the 100-foot passage at an angle of 22 degrees from the normal. It is operated by hydraulic power. When the bridge is carrying traffic it rests upon bearing-blocks at three points under each main girder, the rear set of blocks being fitted with a wedging device. In opening the bridge, the sequence of operations is as follows. The tail end of the bridge is lifted by means of two hydraulic presses to give clearance, and the bearing-blocks are withdrawn. The tail end of the bridge is then lowered until the tail rollers rest upon the roller-path; the nose end of the bridge has risen at the same time and given clearance over the centre and nose end bearing-blocks. The bridge is then rotated through the required angle to bring it parallel with the passage by means of the hydraulic sluing-cylinders, and is finally brought to rest against a hydraulic buffer and stop. The complete operation of opening or closing the bridge is performed in 75 seconds against a wind-pressure of 15 lbs. per square foot. The railway passes over the bridge on the south side of the roadway, but the girders are arranged to enable it to be transferred to the north side if required. The road is paved with jarrah blocks 3 inches wide by 4 inches deep laid on $\frac{3}{4}$-inch asphalt on concrete carried by pressed-steel plating.

The bridge-pivot is carried on a grillage of sixteen 24-inch by 7½-inch rolled-steel joists 12 feet long surmounted by a cast-steel distributing girder 13 feet by 5 feet by 3 feet 3 inches deep. The pivot pedestal-stool and bearing are of cast steel with a bearing-ring of tool steel. The trunnion pin is 1 foot in radius and 3 feet 2 inches long of forged steel with forged-steel crosshead and suspension-links. The cast-steel tail roller-path has a 13-inch tread, and the cast-steel rollers are 3 feet 6 inches in diameter. The centre roller-path is of cast steel with a 9-inch tread, and 3-foot 6-inch diameter cast-steel rollers mounted on spring carriages take the lateral sway. The tail press cylinders are of cast steel with cast-iron rams 1 foot 11½ inches in diameter, the working-pressure being 2,400–3,000 lbs. per square inch. The cast-steel sluing-cylinders are fitted with cast-iron rams 25 inches diameter and 10 feet 1½ inch stroke. The sluing-cables, 10 inches in circumference and 85 feet long, are of flexible plough steel-wire rope with a breaking-strain of 328 tons. All bearing-blocks are of cast steel, the wedging-blocks being operated by 3-inch diameter hydraulic cylinders of solid-drawn steel tube. The hydraulic intensifier for the tail presses is designed with rams 2 feet 1½ inch diameter and 12 inches diameter and 2 feet 9 inches stroke to give the desired maximum pressure of 3,000 lbs. per square inch. The total swinging weight of the bridge is about 1,800 tons, of which 850 tons is kentledge, the preponderance being sufficient to balance a load of 21 tons placed at the extreme nose end. The whole of the concrete foundations of the bridge and operating-machinery are carried down to the ballast. The bridge was erected in position on the west side of the passage by means of a 15-ton Scotch derrick and an 8-ton locomotive crane. It was tested with a train of locomotives and two 15-ton road-rollers, and the maximum deflection in a span of 122 feet 6 inches was 0·3 inch.

Bascule Bridge.—The double-leaf bascule bridge which spans the entrance lock (Figs. 11, Plate 8), is operated by electric power. It carries the public traffic on a roadway 17 feet 6 inches wide between curbs, with overhanging footways 6 feet wide on each side.

Each pivot is set 14 feet 6 inches back from the edge of the lock-coping, giving a 6-foot clear footpath. The pivot-bearings are carried on plate girders spanning the bridge-pit, anchored at the tail end and resting on rocker bearings at the forward end. When carrying traffic the bridge is locked in position by motor-operated bolts at the tail end of each main girder, and also by two bolts connecting the two leaves together in the middle of the span. When opening, the bolts are withdrawn, and the bascules are rotated through the required arc, coming to rest with the aid of air brakes against stops fixed in the bottom of the bridge-pit. In closing, the air brakes and stops take their bearing against the underside of the bearing-girders. Hand gear is provided for operating the bridge in the event of failure of the power. The bridge machinery is designed to open or close the bridge through an angle of 81 degrees, including all operations, in 60 seconds without wind-pressure, and in 75 seconds against a maximum wind-pressure of 15 lbs. per square foot. The road is paved with jarrah blocks 9 inches by 4 inches by 4 inches, laid on 3-inch greenheart planks bedded on 1-inch asphalt; this in turn is laid on concrete carried by pressed-steel plating. The blocks are secured to the planking by 5-inch galvanized coach-screws at 2-foot centres, and each block is dowelled with oak pins on all four sides. The cast-steel pedestals of the main bearings are mounted in 14-inch diameter phosphor-bronze bearings fitted with forced lubrication. The main pivot, 15½ inches diameter by 2 feet 7 inches long, is secured to the cast-steel inner bearing of the main girder by a $2\frac{1}{16}$-inch through bolt. The cast-steel rocker-bearings have 5-inch manganese-steel pins. The gearing and rack-segments are of cast steel. The nose-end bolts are 4 inches in diameter by 9 inches stroke, and the tail bolts are 3½ inches in diameter by 6 inches stroke. The time allowed for housing or withdrawing all the bolts is 6 seconds. Each bolt is operated by a 2-HP. motor. The cast-steel air-brake cylinders are 14 inches in diameter and 1 foot 7 inches stroke. The 50-HP. main motors for each leaf are installed in duplicate and are ordinarily used together, but either can be uncoupled when required. The speed of the bridge is controlled by magnetic brakes. The operations of the motors for each leaf are electrically interlocked, and an automatic indicator is fitted in the operator's cabin.

The total weight of the bridge is 742 tons, including 361 tons of counterweight. The concrete foundations are carried down to the chalk. The bridge was erected in position by means of a 15-ton Scotch derrick on each side of the lock, without the use of any falsework. It was tested with a moving load of road-rollers and trollies, the maximum deflection being 0·66 inch.

Penstocks, Capstans, etc.—A pair of wall penstocks operated by hydraulic power is provided at each pair of gates in the entrance lock, while at the outer gates there is also a pair operated by hand gear for use in emergency. At the dry dock, again, each of the two filling-culverts is provided with a hydraulic and a hand-gear penstock. The greenheart paddles in all cases are built up of logs 11 inches square, bolted together with strong through bolts and fitted with lifting-eyes. At the entrance lock the cast-steel hydraulic cylinders are 24 inches diameter and 11 feet 3 inches stroke, fitted with ⅜-inch gun-metal liners. The pistons are of cast iron with hemp packing; and 5½-inch copper-coated piston-rods are cottered to the cast-iron crossheads, each of which is fitted with 3-inch square steel lifting-bars pinned to brackets on the top of the paddles. At the dry dock the construction is of a similar character, but the cylinders are 12 inches in diameter and 6 feet 9 inches stroke, with 3½-inch diameter copper-coated rods; heads with hand spikes operate a screw through gearing. The screw is 6½ inches in diameter at the entrance lock and 4 inches in diameter at the dry dock, the pitch in both cases being ⅝ inch.

Fifteen hydraulic capstans are provided, of which eight are at the entrance lock, four at the passage to the Royal Albert dock, and three at the dry dock. They are all of one pattern, designed to exert a pull of 10 tons at a maximum speed of 80 feet per minute. The barrel is 18 inches in diameter. Each capstan has three cast-steel cylinders with 6½-inch cast-iron rams, 12 inches stroke. The crankshaft is 6 inches in diameter, and the gearing is of cast steel with a 3-to-1 reduction. The two capstans on the timber entrance-jetties are housed in steel-plate tanks below the deck-level. Checking-bollards of the post type are used at the entrance lock, dry dock, and passage, and mooring-bollards of the double horned type are provided at the quays. The checking-bollards are 10 feet 3 inches long, standing 3 feet 3 inches above ground-level, and are 18 inches in diameter, set back at an angle of 12½ degrees from the vertical. At the entrance lock they are spaced at 50-foot centres, set 20 feet back from the edge of the coping, and are carried on concrete counterforts built up with the main wall. At the dry dock they are spaced at 80-foot centres, being set back 35 feet from the coping on special concrete foundations. The mooring-bollards are kept as small as possible consistent with strength, and they stand only 16 inches above the coping and 16½ inches from the face of the quay-wall, thus enabling the cranes on the quay to be kept close to their work and to be unimpeded by mooring-ropes. Smaller bollards of the same type are used in the barge passage at the jetties. The fairleads, fixed near the capstans, are 18 inches in diameter at the bottom of the groove and are carried on tapered steel pins with 6-inch spherical heads which permit a slight angle of swivel to suit the direction of the ship's rope. Cast-iron ladders and hitcher boxes are built flush in the walls of the lock, dock, and passage at suitable intervals, the hitcher boxes being used to facilitate the movement of barges. Removable fog chains and stanchions are provided at all dangerous points.

Hydraulic Pumping-Station.—The existing hydraulic installation at the Royal Albert dock was of insufficient size to carry the additional load due to the dock-extension, and a new pumping-station was therefore erected as an extension of the impounding-station near the lower Gallions entrance lock. Two multi-stage turbine pumps, each with a capacity of 400 gallons per minute at a pressure of 800 lbs. per square inch, were installed. The pumps are directly coupled to 370-HP. two-phase motors with a normal frequency of 50, the pressure being 6,000–6,600 volts and the speed 3,000 revolutions per minute. The pumps are balanced against end thrust and are mounted on nickel-steel shafts with bronze impellers. The casings are of cast iron, and the flexible couplings are of cast steel. Two $8\frac{3}{4}$-HP. centrifugal storage-pumps raise the water from the settling-tanks to a storage-tank on the roof. Two 6-inch Kent venturi-meter tubes are fitted, one to the suction of each main pump, with automatic diagram counters and pressure-recorders. An additional 6-inch hydraulic main is laid from the station to the dock-extension, and two new 20-inch by 24-foot accumulators are provided near the inner end of the new entrance lock.

Roads.—Over 3 miles of macadam road were constructed in connection with the new dock, the widths inclusive of footpaths ranging from 30 to 50 feet. These have to carry heavy traffic and are generally laid on comparatively new filling. Ultimately, when settlement is complete, it will no doubt be an advantage to substitute more durable paving. The dock roads have a bed of broken stone or broken concrete 12 inches thick covered with 6 inches of broken granite having a tar-sprayed surface. The lower end of the diversion of Woolwich Manor Way runs in a cutting through an old mud-field of deposited mud and refuse, and, in this case, the excavation was taken out to a depth of 4 feet below the road-formation and filled with ballast excavated from the site; this served to distribute the pressure and has given a satisfactory result.

About 11 miles of new railway-track, with the necessary crossings and turnouts, have been laid. The rails are of the 75-lb. flat-bottomed British Standard section, laid on sleepers at 2-foot 9-inch centres, or on reinforced-concrete foundations to quays and paved areas. The minimum radius of curves is 7 chains for running-roads, and 5 chains for sidings. The crane-tracks are laid with similar flat-bottomed rails.

Drainage.—Separate systems for surface water and soil drainage have been adopted. The surface water is drained directly into the dock from shed-roofs, quays, etc., where the necessary fall can be obtained. For the roads and cart areas on the south side of the dock the surface water is led into the watercourses maintained by the Sewer Commissioners, and in this connection the Port Authority had to provide a new outlet into the river in substitution for the former outlet, which lay to the north of the new entrance lock. For the new outlet a 3-foot reinforced-concrete culvert was laid for a length of 400 yards along the High Street, discharging into the river near North Woolwich Station, where the usual penstock and non-return flap-valve are provided. On the north side of the new dock, where a gravity outlet was not available, it was necessary to pump; and two sets of 4-inch Stereophagus pumps were installed. The pumps are electrically driven by 4-HP. motors running on continuous current at 460 volts; and each pump, controlled by automatic float gear, is capable of dealing with 200 to 275 gallons per minute against a total head of 20 feet. The separate soil-drainage system is laid in cast-iron pipes with connections to the local sewers at suitable points, the only special feature being the provision for pumping the sewage under the passage and the entrance lock, where duplicate sets of 3-inch Stereophagus pumps are installed with automatic float-starters.

The dredging of the passage to the Royal Albert dock and of the river entrance to the lock was carried out by the Port Authority's dredging-plant. About 1,000,000 cubic yards of material were removed in this way.

For filling the new dock a flume was constructed 8 feet wide, communicating with the Royal Albert dock and discharging on the abandoned railway-bank in the north-west corner of the new dock (Fig. 12, Plate 8). The water-level in the Royal Albert dock is maintained by impounding-pumps at or about 2 feet 6 inches above Trinity high water. The bottom of the flume was 2 feet below the impounded level. It was desired to commence filling the dock as soon as the inner pair of lock-gates was finished, without waiting for the completion of the middle and outer gates, which were in course of erection in the dry. The middle and outer gates were completed 5 and 10 months later respectively. It was not considered wise to allow the full hydrostatic pressure due to a filled dock to bear upon the invert and walls on the lock, and arrangements were therefore made to reduce the head to about 20 feet by providing two earth dams in the lock as shewn in Fig. 12, Plate 8, and by flooding to the required depth over the invert. This enabled the gate-platforms to be kept dry until the other two sets of gates were completed. Before commencing filling the segmental concrete dam at the passage was removed.

On the 12th January, 1920, the Chairman of the Port Authority, Lord Devonport, opened the electrically-operated sluice, and the filling proceeded at the rate of about 1 foot per day up to O.D. The middle pair of gates was closed on the 22nd May, 1920, and, the adjoining earth dam in the inner compartment having been removed, the filling was continued from the 24th June and was finally completed on the 12th August, 1920; from that date onwards a uniform level was maintained in the two docks, a breach having been dredged through the earth-bank at the passage, which was opened for craft on the 21st August. The outer pair of dock-gates was closed on the 5th November, and, the adjoining earth dam having been removed, the dredging of the river entrance channel was commenced. On the 21st February, 1921, the Port Authority's steam yacht "Carmela," with the Chairman and Vice-Chairman on board, passed from the river through the entrance lock into the new dock.

EQUIPMENT OF DOCK.

Sixty-six quay-cranes are provided for handling cargo ; these are disposed six to each of the seven jetties, the remaining twenty-four being on the north quay. Two of these cranes are the special cantilever type designed by Sir Cyril Kirkpatrick, and the remaining sixty-four cranes are of the usual type of self-propelling pedestal jib-cranes. They are electrically operated, working at 460 volts continuous current, and are fitted with level-luffing gear. The leading particulars are as follow :—

Gauge, centres of rails	13 feet 6 inches.
Maximum radius, 3-ton load	60 feet.
„ „ reduced load	65 „
Minimum „	20 „
Total lift at maximum radius	115 „
Speed of lifting, 3 tons	175 feet per minute.
„ „ 1½ ton	250 „ „
Speed of sluing, full load at maximum radius .	1 revolution per minute.
Speed of luffing	2 feet per second.
Travelling with full load at maximum radius .	100 feet per minute.
Weight of crane	60 tons (approx.).

The two special cranes are designed to meet the same general requirements as the ordinary quay-cranes, but to give quicker working and a more accurate control. The general design comprises a double cantilever revolving on a pedestal and carrying a small traversing jib-crane. The leading particulars are the same as those given above, excepting that the speed of traversing of the small jib-crane is 200 feet per minute, and it slues 2 revolutions per minute. The weight of the crane is approximately 100 tons. In addition to the quay-cranes, fourteen runabout electric cranes are provided for use in the sheds. These lift 15 cwt. at a radius of 10 feet to a height of 16 feet. The speeds per minute are : hoisting, 60 feet ; travelling, 50 feet ; sluing, 3 revolutions. The weight of each crane is 5 tons. For special service, where quay-cranes are not available, a 3-ton locomotive crane on road wheels, driven by a paraffin motor, has been provided. The radius is 38 feet, and the speeds are : 100 feet per minute hoisting and travelling ; 1 revolution per minute sluing. The weight is about 30 tons.

MATERIALS.

Over 1,000,000 cubic yards of sand and ballast were excavated from the site. About one-half of the total quantity was used for concrete, the remainder being filled in behind the quay-walls and utilized as a top layer of filling to the reclamation areas. The proportion of sand found in the ballast varied considerably and was usually in excess of requirements for obtaining the best concrete. The average composition of the natural ballast was as follows :—

(*a*)	(*b*)	(*c*)	(*d*)	(*e*)	
Volume of Ballast.	Volume of Shingle (measured separately).	Interstices in Shingle.	Volume of Sand (measured separately).	Interstices in Sand.	Cubic Feet of Sand to Cubic Yard of Ballast.
1	0·69	0·34 (*b*)	0·40	0·36 (*d*)	10·8

Weight per cubic foot (dry) :—

Ballast	116 lbs.
Shingle	104 „
Sand	95 „

The cement was specified to be "medium" setting, and in accordance with the British Standard specification. About 100,000 tons of cement was used on the works, the greater portion being obtained from the Wouldham Cement Company, whose deliveries easily met the requirements of the specification. The "medium" setting quality was not insisted upon, and slow-setting cement was used almost exclusively. Storage was provided on the site for 2,000 tons, but most of the cement was delivered straight to the concrete-mixers.

Careful experiments were carried out to determine the strength, weight, and porosity of concrete with varying proportions of sand. The best results were obtained with a mixture of sand and shingle giving an excess of mortar over the interstices in the screened aggregate of approximately 15 per cent. The following proportions were finally decided upon and adhered to throughout the work :—

Quality of Concrete.	Each Cubic Yard of Ballast to Contain	Proportions.		
		Cement.	Sand.	Shingle.
3 to 1	6¾ cubic feet of sand	1	¾	2·6
4 to 1	8½ " "	1	1¼	3·2
6 to 1	9 " "	1	2	4·7
8 to 1	9¾ " "	1	2·9	6·1

Crushing tests of 12-inch cubes were made with the following results :—

Quality of Concrete.	Average Crushing Load at 3 Months.
3 to 1	220 tons per sq. ft.
4 to 1	160 " "
6 to 1	147 " "
8 to 1	103 " "

The strength increased with age up to 12 months.

Percolation tests were made on 12-inch cubes under a head of water of 35 feet. The percolation decreased with time with all qualities of concrete with tests continued up to 90 days, showing the gradual closing up of the pores of the concrete. This is shown by the typical diagram, *Fig. 13.* The average percolation after 20 days under test is shown in the following Table.

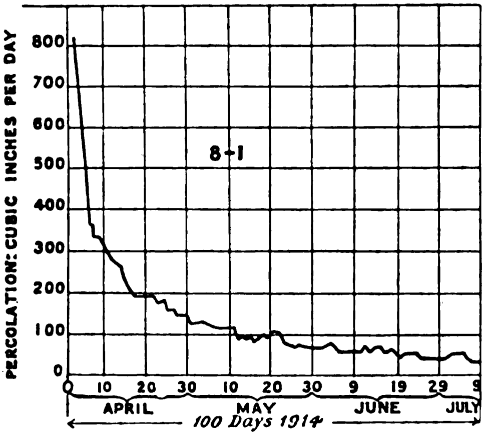
Quality of Concrete.	Average Percolation in Cubic Inches per Day.
3 to 1	1·26
4 to 1	20·0
6 to 1	46·0
8 to 1	180·0

The aggregate used for reinforced concrete was composed of clean crushed ballast, the stones being of assorted sizes not less than ⅛-inch gauge and not more than ¾-inch gauge. The concrete was mixed in the following proportions :—

Cement. Lbs.	Sand. Cubic Feet.	Aggregate. Cubic Feet.
224	3⅜	6¾

The whole of the granite ashlar was obtained from Cornwall, being generally delivered on the site dressed to finished sizes. Very little mason's work was done on the site, with the exception of dressing watertight faces, cutting bolt-holes, etc.

Fig. 13.



Temporary Works.—The general layout of the plant and method of operation is indicated in Fig. 12, Plate 8. Great credit is due to the contractors for the excellency of their plant, which was used not only during the contract period, but also for the completion of the works by direct administration.

Electric power, supplied in bulk by the West Ham Corporation at a pressure of 6,600 volts, was used wherever possible, after suitable transforming, for operating pumps, concrete-mixers, etc., but its use was not extended to cranes, excavators, or navvies. Transformer-houses were installed at convenient points from time to time, and the arrangements worked very satisfactorily. Occasional inconvenience arose due to the cutting off of the supply for short periods, which caused the stoppage of pumps and other machinery, but such interruptions were few and of short duration. The works were carried out in the dry, and the positions of the more important pumping-sumps are indicated in Fig. 12, Plate 8. The centrifugal-pump with vertical spindle was adopted as standard, the output of each being 1,000 gallons per minute at 40 HP.; and these pumps were usually slung from the timbering in such manner that they could be lifted out of danger. They were easily transferred to different positions to suit the progress of the work, and proved in every way more convenient than heavy fixed pumps. Over long periods the total pumping averaged about 10,000 gallons per minute with a maximum of 13,000 gallons per minute. The natural ballast in situ was very close and impervious to low pressures, and the water from deep foundations was often discharged by small pumps into grips in the ballast on the dock-bottom to drain away to the main sumps.

The specification provided for the general excavations of the dock to be done by dredging or otherwise after the completion of the quay-walls in timbered trenches. The contractors, however, arranged for the dock-excavation to be commenced at the outset, to enable the ballast to be uncovered in readiness for the concrete walls. Two Lübecker land-dredgers were successfully employed for this purpose. Each machine weighed about 130 tons and was carried on 90-lb. rails at 15-foot 9½-inch centres, the rails being supported on 12-inch by 6-inch sleepers. The ladders carried thirty-five buckets of ¼-cubic yard capacity. The engines were of 120 HP., and the depth of excavation down to the ballast was about 20 feet, the average cutting slope being about 2½ to 1. Each machine had in attendance two locomotives and about 120 end-tip 10-ton wagons. The output of each machine and attendant plant under favourable circumstances was about 2,000 cubic yards per day in situ. Fig. 12 shows how the Lübeckers were advanced from south to north by sluing the tracks. Subsequently one of them was employed in carrying the excavation parallel and close to the western side of the old Manor Way Road. Heavy machines of this type can only be employed successfully on reasonably good ground, and in dealing with silted-up watercourses like the Old Ham creek, it was necessary to use a navvy working on the ballast-level to remove the soft silt. Steam navvies were employed for the excavation of the top spoil in confined spaces on the site of the entrance lock, the dry dock, and passage, and for the removal of the old Manor Way Road after traffic had been diverted.

At the entrance lock a Ruston 18-ton steam navvy, weighing about 52 tons, with a 2¼-cubic-yard bucket, was employed with an output of about 1,500 cubic yards in situ per 10-hour day. At the dry dock a Whittaker 12-ton steam navvy, weighing about 35 tons, with a 1¼-cubic-yard bucket, was employed with an output of about 1,100 cubic yards per day. A 12-ton Wilson steam navvy with a similar output was used for the removal of the old Manor Way Road. A small Marion steam shovel, weighing about 12 tons, with a ¾-cubic-yard bucket, on a caterpillar track, was used for levelling the ground on the south side of the entrance lock. It was worked over very soft ground on a sleeper road and proved very adaptable, giving an output of about 400 cubic yards per day. Priestman grabs were largely used in excavating the ballast over the whole site, and for trench excavation. Part of the clean ballast was screened and used for securing the proper proportions for making concrete.

About 17 miles of 4-foot 8½-inch gauge temporary railway were laid over the site. Sixteen locomotives were employed, mostly six-wheeled with 12-inch to 14-inch diameter cylinders. About two hundred and sixty 4½-cubic-yard side-tip wagons were in use for filling and tipping to spoil, etc. About three hundred and sixty 10-ton end-tip wagons, with a capacity of 15 cubic yards, were employed for carrying the excavation to the hoppers to be disposed of at sea. Such of the excavations as were not required were sent to sea a distance of about 60 miles in steam or in dumb hoppers loaded by the two tipping-hoists in the Albert dock. The gradient of the track from the dock-bottom was 1 in 30, and the approach to the hoists was upon trestles, the gradient here being 1 in 60. Each hoist had tracks for full and for empty wagons. The Port Authority's and Contractors' railways were bridged by the viaduct. To clear the railway-traffic to the hoists it was necessary to divert the main road giving access to the south side of the Royal Albert dock; and this was carried on a temporary bridge built up of timber piles and trestles, with steel girders supported by earth-banks at each end. The temporary road was 23 feet wide with a maximum gradient of 1 in 50.

The two tipping-hoists were designed to lift 10-ton trucks full of spoil 30 feet above quay-level. The steel towers were 80 feet high and 24 feet by 24 feet in plan, placed 33 feet apart between centres. They were operated by hydraulic power at 750 lbs. per square inch, the pressure-pumps and accumulators being housed near the hoists. Three sets of 65-HP. diagonal pumps were driven by gearing from alternating-current motors. Steam winches were used for hauling the trucks to and from the hoists, and a 5-inch centrifugal pump was used for washing and lubricating the shoots for the spoil. Each hoist tipped an average of 40 trucks per hour in regular work.

The 1,200-I.HP. steam hoppers were each of 1,000-cubic-yard (or 1,250-ton) capacity, with a speed of 11 knots, and averaged 10 hours for the round trip under fair conditions. The dumb hoppers were each of 1,200-cubic-yard capacity and were usually towed by the tug "Lady Curzon," making the round trip in 16 hours under average conditions.

Of the total excavation of about 5,000,000 cubic yards, over 2,000,000 cubic yards were deposited at sea; under fair conditions the hoppers carried about 30,000 cubic yards per week.

The concrete-mixers in general use for the mass concrete were of 1½-cubic yard capacity, and they were mounted on 4-foot 8½-inch gauge bogies for easy transport. They were driven by 20-HP. alternating-current motors and were fitted with platforms and measuring-hoppers for the ballast, which was usually fed to the mixer by a grab. Seven of these large mixers were in use, most of them being Ransome mixers. Small mixers were used for reinforced concrete, usually of ¾-cubic yard capacity.

Panel shuttering of 3-inch timber was used for building the concrete walls. Each panel was 14 feet by 7 feet, giving 6 feet effective height. The concrete was deposited in two layers of 3 feet each. The shutters were secured by bolts screwed into plates buried and left in the concrete, and by timber ties on the top between the front and back shutters. Very little trouble was experienced through the movement of the shuttering under the pressure of the concrete, and the finished appearance of the face-work was very good.

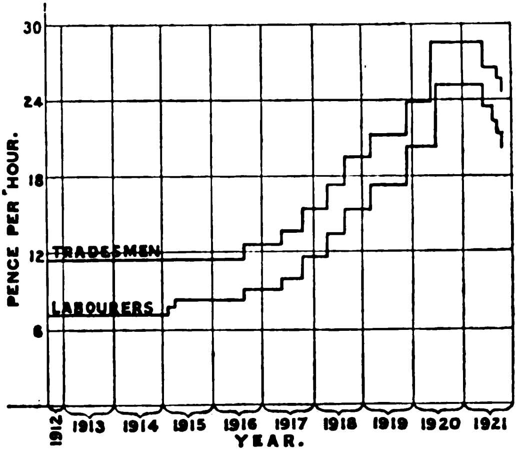
Over thirty cranes were in use on the site. These were mostly steam locomotive-cranes, ranging from 5 to 10 tons capacity, with provision for working either on the 4-foot 8½-inch or the 7-foot gauge as required.

Fig. 12, Plate 8, shows the position of the repair shops, which comprised engineers' fitting- and machine-shops, smiths', boiler-makers', and electricians' shops, foundry, locomotive shed, sawmills, and carpenters' and wagon-repair shops, with the necessary stores, etc. They were equipped with electric power and proved invaluable for the efficient carrying on of the work.

Over 3,000 reinforced-concrete piles were made on the site. Piles and other pre-moulded work for the dock-jetties were made on the bottom of the dock, and piles for the double-storey sheds were made on the north quay. In both cases ample areas were available as stacking grounds, which were commanded by cranes and had railway access. The ballast for reinforced concrete was crushed, washed, and screened on the site.

The diversion of Manor Way over the new entrance lock had to be effected at the earliest possible date to enable the old road to be excavated and the quay-walls to be completed. As the bascule bridge over the lock could not be completed in time, a temporary bridge was constructed immediately to the westward, using the Goliath girders from the lock. This temporary bridge carried the public traffic for over 2 years and was cleared away as soon as the bascule bridge was finished.

Fig. 14.

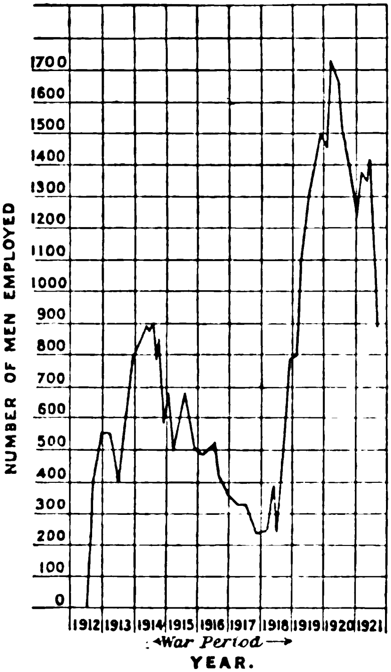


The total cost of the dock and its equipment as at present authorized is about £4,130,000. A summary of the equipment is given in the Appendix. The cost is spread over the period 1912–1922. Owing to the great fluctuation in prices of labour and materials during that period, the actual expenditure in connection with the dock cannot be presented as being representative of standard values for similar works. The fluctuation of rates of pay of labourers and tradesmen in the building trades during the period of construction is shown in *Fig. 14.*

The number of men employed on the site over the whole period of construction is shown by *Fig. 15.* It will be observed how pronounced was the effect of the war in delaying the completion of the work. Two strikes of short duration took place, one in 1913 and the other in 1920, but the aggregate stoppage on this account was little more than 1 month.

No serious accident took place throughout the whole period of construction due to failure of plant or temporary works. Unfortunately twelve men lost their lives in the 9 years during which operations were in progress, all of them being isolated cases due to simple individual causes.

Fig. 15.



The Rt. Hon. Viscount Devonport, P.C., was Chairman of the Port Authority from the inception to the completion of the works. Mr. Palmer was responsible for the carrying out of the works, as Chief Engineer, up to the time of his resignation in 1913, and thereafter, as Consulting Engineer to the Authority, up to the time when the contract was terminated. The works were then carried to completion under the direction of Sir Cyril Kirkpatrick, M. Inst. C.E., the present Chief Engineer. The Author was Resident Engineer throughout the design and construction of the work, Mr. J. H. Walker, M. Inst. C.E., being his chief assistant.

On behalf of Messrs S. Pearson & Son, Ltd., the main contractors, Sir Ernest W. Moir, Bart., M. Inst. C.E., was the director in charge, and Mr. F. W. Duckham, M. Inst. C.E., was the agent on the site. On behalf of Sir William Arrol & Co., Ltd., contractors for the lock-gates, caisson, bascule and swing bridges, etc., Sir John Hunter, K.B.E., was the director in charge. Messrs. George Corderoy and Co., were the Port Authority's quantity surveyors throughout the work.

The dock was formally opened by His Majesty the King, accompanied by Her Majesty the Queen, on the 8th July, 1921, and was named "The King George V Dock."

The Paper is accompanied by seventeen sheets of diagrams, from some of which Plates 7 and 8 and the Figures in the text have been prepared.

APPENDIX.

STATISTICAL.

Wet Dock.

Water area	64 acres.
Quayage	2⅓ miles.
Berthing accommodation	14 large vessels.
Depth of water	38 feet.
Pumped level above T.H.W.	2½ feet.

Entrance-Lock.

Length	800 feet.
Width	100 feet.
Depth over sill below T.H.W.	45 feet.
Gates	3 pairs.

Dry Dock.

Length	750 feet.
Width	100 feet.
Depth (over keel blocks)	35 feet.

Passage between Royal Albert Dock and New Dock.

Width	100 feet.
Depth	34 feet.

Single-Storey Sheds.

Seven. 520 feet × 120 feet (2 spans, 60 feet).

Double-Storey Sheds.

Two. 1,100 feet × 120 feet (ground floor), × 110 feet (first floor).
One. 1,150 feet × 120 feet (ground floor), × 110 feet (first floor).

Roads.

Over 3 miles, 30 to 50 feet wide.

Railways.

11 miles, standard gauge.

Cranes.

66	3-ton electric travelling cranes,
14	15-cwt. electric runabout cranes,
24	1-ton overhead cranes,
12	1-ton wall cranes,
1	25-ton electric travelling crane,
1	portable petrol driven crane,
1	5-ton steam crane.

Quantities.

Excavation	approximately	5,000,000	cubic yards.
Dredging	,,	1,000,000	,, ,,
Concrete	,,	500,000	,, ,,
Granite	,,	250,000	cubic feet.

Discussion.

The **President** moved a vote of thanks to the Author. The President.

The **Author** exhibited a number of lantern-slides showing the construction of the works. The Author

Mr. **Frederick Palmer** thanked the Author for his excellent Paper, which was of particular interest to him personally, as he had been closely associated with the initiation of the work described. When the Port of London Authority came into existence, it was found that powers existed for the construction of a dock to the south of the Royal Albert Dock; but the dimensions which had been planned, many years before, seemed so inadequate for modern requirements, that it was decided to proceed on a very much larger scale. That necessitated the purchase of a great deal more land by the Port Authority, including the houses referred to by the Author, and resulted in a dock of the fine dimensions described in the Paper. The Author had done him the honour of referring to him as the designer of the dock, but he would like to associate with that work almost the whole of the then somewhat scanty engineering staff engaged under the Port Authority, including Sir Cyril Kirkpatrick, the present Chief Engineer; and in particular he desired to mention the great assistance given by the Author, who had been fortunate in being associated with the work from its inception to its completion. The introduction of the jetties along the south wall was largely an experiment. In the Port of London so much of the merchandise was taken away from the docks by lighters that there had always been a demand for better facilities for lighter-borne traffic; and the design in question, which allowed barges to get on either side of vessels, had been evolved to meet that requirement. It was probably too early yet to say whether the innovation had been as successful as was hoped; perhaps the Author would give his views in that regard as a result of his experience at the dock since its opening, nearly 2 years ago. At all events, it was not a costly experiment, because the saving effected in building the south quay-wall to a less depth must have gone a long way towards meeting the expense incurred upon the jetties. It had been originally intended to build the vents in the floor of the dry dock, but, in getting out the foundations, a bed of Thanet sand was met with at the bottom, and it was feared that water forced through Mr. Palmer.

Mr. Palmer. the vents might bring up so much of the sand as, possibly, to let down the walls or the floor. It was therefore decided to put the vents above the bilge altar, where they were backed by the ballast found in such quantities and of such excellent quality over the whole site. The Author rightly called attention to the excellence of the plant employed by the contractors and to their methods of carrying out the work. The excavation of the spoil by the Lübecker land-dredgers and its disposal had been quickly done, and the gantries used in the construction of the entrance-lock were cleverly designed. One feature of the design, not often employed in such work, was the provision of ample subways under the entrance-lock. In conclusion, he desired to mention that the conduct of the work by the Author had been excellent in every way. In addition to the ability with which he had carried out his work as resident engineer, he displayed that fine tact which covered the firmest of hands with the softest of velvet. It was very gratifying to Mr. Palmer to have been associated with the Author for so many years, and to have had work for the design of which he was responsible carried out to such a successful conclusion under the difficult conditions brought about by the war.

Mr. Moncrieff. Mr. J. MITCHELL MONCRIEFF remarked that he had read the Paper through several times with great interest, and each time had felt a desire for more information: the work described was so large and contained so many items of interest. In 1917, when Director of Engineering Work at the Admiralty, he had the pleasure of seeing the dock, under the Author's guidance, during its construction. In reading the Paper, his mind went back to those days, and he recalled the conditions which prevailed at that time and the class of construction that was going on; but he felt that the Paper could not convey that, in a general sense, unless more information was given. The lantern-slides exhibited that evening had answered most of the questions he had desired to ask. Members who were not present, however, would not be able to understand what those who had seen the photographs had grasped so easily; and he hoped, therefore, that the Author would be able to add to the Paper. From time to time, in reading it, one came to a point where one expected certain information which was not given. For example, the sections of the dock-walls were given, but there was nothing to indicate what the walls had to carry. It was not stated whether the ballast started at the bottom of the dock or half-way up. It would be interesting to know the class of earth and the conditions met with in that earth. It must have been gratifying for the contractors to come across such fine ballast. He noticed that the sill of the dry

Mr. Moncrieff.

dock was only 3 feet above the floor, while the keel-blocks were 4 feet high. He always liked to keep the top of the blocks a little below the level of the sill at the entrance, so that an incoming vessel should not have any opportunity of overturning them, It was only natural that a dock-master should think of the depth of water on his sill; he might remember to think of his blocks, but there was always a danger of their being overturned if they stood above the sill. The jetties in the main dock were interesting, and he would like to have more information about certain features of them. The Author, in referring to the lower back braces, said they were pre-moulded, with an eyelet slipped over the pile. Only one eyelet was spoken of, but the piles were battered, and he could not see how they had been got on. That was an interesting piece of construction, and he would like to know how the other end had been dealt with. Again, the jetties were very tall. They had to accommodate, he supposed, any vessels that could enter the dock. It was true that, once inside the dock, the vessels were in still water; but wind had to be considered, and a vessel in motion was difficult to stop. The jetties appeared to be light, and the piles were not driven very deeply, which again made him wonder what kind of ground existed there. A very large number of concrete piles had been driven, and considerable experience must have been gained in connection with driving; he would like to know whether any peculiar circumstances had been met with. He had encountered some curious results in connection with the heavy driving of concrete piles; horizontal cracks spaced at about 9 inches to 1 foot apart had developed over quite considerable lengths of pile. That was with fairly heavy driving, where the piles were driven much deeper than in the case described. The Author had had the opportunity of making some extremely interesting experiments on materials, but the account of them was not complete. In particular, in connection with the percolation-tests on 12-inch concrete cubes, he would like to know the area over which water-pressure had been applied to the cube. How had the experiments been carried out? *Fig. 13* was one of the most interesting diagrams he had seen in connection with percolation, and he was sorry that such diagrams were not given for other mixtures of concrete. Was the water salt or fresh, and why did the pores close up? He was particularly interested in that, because he had had under observation for a good many years certain concrete works in sea-water, which had undoubtedly swollen and broken. The cause of that was known, but he would like to know what the material was that was deposited during the 90 days in the test-blocks referred to by the Author.

Mr. Deane. Mr. H. J. Deane associated himself with what Mr. Moncrieff said about the brevity of the Paper. The work possessed interesting features, concerning which further particulars would be welcome. When the bascule bridge was first designed, there were comparatively small clearances between the nose ends of the bridge and the sides of the lock when the bascules were open, but, on account of the construction of vessels with overhanging bridges, it was necessary to increase the length of the racks, so that the bridge could now be opened to 6 feet beyond the coping. That also raised a question in regard to the jetties and the cranes on them. The cranes were designed so as to allow 2 feet 6 inches clearance back from the face of the jetties. It was obvious, however, that with a ship's bridge overhanging 5 feet the cranes would be swept off the jetties, and in consequence wide timber booms had to be employed to prevent the ships from colliding with the cranes. Dock-engineers should bear in mind that ships were already being built with an overhang of 5 feet, and in future the overhang might be more than that; so that existing docks might prove unsuitable on account of the smallness of the clearance allowed. Two classes of pumps had been installed, the 5-inch hydraulic turbine pumps at the Gallions entrance for hydraulic work, and centrifugals for emptying the dry dock. Both classes were driven by synchronous motors, which were started as induction motors, and, as soon as they reached their correct speed, were run on the synchronous windings. At the speed required to obtain the necessary head a large portion of the full output was absorbed. The ordinary type of synchronous motor could only exert a very small percentage of its full output at the time of change-over. The 5-inch hydraulic turbine pumps took 41 per cent. of the working-load, and the de-watering pumps 33 per cent. at the time of the change-over; and it was necessary that pump- and motor-makers should appreciate fully that requirement, and design their motors accordingly. The question had been raised whether the water was fresh or salt. Observations of the specific gravity of the water had been made, the results of which were interesting. Caissons had been built for Tilbury Dock, the King George V Dock, and Millwall Docks, and the specific gravity of the water had been found to be:—Tilbury, 1·017; King George V, 1·006; Millwall, 1·002. That showed that as one went up the river the water became fresher, and therefore considerably lighter, so that the flotation of the caissons had to be designed accordingly. Another point of interest, to which the Author had not referred, related to the swing bridge. That bridge, as originally designed, was mounted on a "tabloid" pivot. The tabloid was provided with spherical upper and lower surfaces, the upper working in a spherical cup under the bridge, and the lower resting in a similar cup on the foundation casting. That type of design was very suitable for the operation of swing bridges where the sluing force was applied on diametrically opposite points of the racks; but, where the load was applied by means of a single rope or chain, a stress was exerted which displaced the bridge, causing a slight lift and a considerable eccentricity. In the case of the bridge in question, weighing 1,800 tons, the pull of the one ram which slued the bridge was 180 tons, and it would be found by calculation that the total eccentricity obtainable would have been about 7 inches, corresponding with the comparatively small rise of about $\frac{3}{4}$ inch. It was not possible to avoid movement by putting a ring round the tabloid, so the pivot had to be redesigned. Mr. Deane.

Sir Ernest Moir.

Sir ERNEST MOIR observed that it gave him great pleasure to be present at the reading of the Paper. Before the war, and for some 4 years after the war, his firm had been engaged, together with the Author, in carrying out the work described, and the association had been very pleasant. It had been his privilege to carry out dock-construction work in the Thames for four different engineers in connection with what was now the Port of London Authority—Mr. James McConnochie, Sir John Wolfe Barry, Mr. Frederick Palmer, and, succeeding him, Sir Cyril Kirkpatrick. He had, therefore, enjoyed the exceptional advantage of seeing the line of thought, put into material, which had actuated all those engineers in design. He remembered, in connection with the Surrey dock, Mr. McConnochie saying that he was "not there to build monuments, but to use material and construction that would produce dividends." Personally, he thought that was a very fine basic idea to work upon for any engineer who was designing productive undertakings. In constructing the Greenland dock entrance—part of the old Surrey dock—one of the pre-historic locks, the structure of which was founded on timber, had been cut out by his firm. The clapping-sill was of oak, resting on piles of similar wood, and the lock had been there for so long that no one knew who built it or anything about its design. It had been carrying out its duties faithfully all that time without any mishap and with but little repair. He sometimes thought that, not perhaps intentionally, monuments were put up instead of works to produce dividends, and it was a fact that dock-structures were generally out of date before they were worn out. It was noticeable abroad, at, for instance, such a port as New York, that even to-day the largest liners were being brought alongside timber piers. He had seen hundreds of piles, with not even the bark taken off the timber, put in on the Hudson River. It might

Sir Ernest Moir. be that in such cases maintenance charges would be too heavy; but there was an economic dividing line between maintenance charges and capital outlay. It was stated in the Paper, for instance, that certain cubes of concrete had withstood 103 tons per square foot crushing load, and it was also stated elsewhere in the Paper that the gravel on which that concrete was placed was only required to stand a pressure of 5 tons per square foot. Cement was being constantly improved, but the proportion of cement to gravel had not generally been reduced. Greater fineness of grinding, greater density, and greater strength when set, were specified, but the old gross ingredients were still generally adhered to. The question should be considered whether, if concrete was available which would bear 103 tons per square foot, standing on gravel which would only carry 5 tons, the strength of the concrete might not be reduced somewhat by more economical use of cement. It was not often that the contractor had an opportunity of criticizing the engineer—the boot was usually on the other leg—so perhaps he might be excused for these remarks. One of the principal possible difficulties to be guarded against arose in connection with the construction of the dry dock. The Thanet sand on which it was founded, where water had been allowed to get into it, presented considerable obstacles and dangers while being trenched for the walls. To overcome this a series of sumps had been put down round the dry-dock site long before excavation was commenced, so that the material might give up its water, and so that the gravel above it might be drained and not be ready at all times to discharge through the Thanet sand the water which it contained. When the excavation was started, the Thanet sand was dry, and no trouble was met with. The pumping of such a large area, surrounded by the docks and the river, presented difficulties of an unusual kind. Another thing which caused some trouble was that mud from maintenance dredging had been pumped ashore from the Millwall dock for many years and deposited on the site of the main entrance. The mud was hard on the surface, but at a depth of 15 feet it was very nearly as liquid as when first placed there many years ago. The only complaint he had against the Paper was that it gave too much information—of a somewhat sanguine character at times. The information given was almost sufficient to show what the contractor's costs were; but in that connection it should be borne in mind that the outputs given for the various pieces of plant did not represent the average achieved, but the maximum. That the plant would do what the Author stated was, no doubt, true, but that it did it always was not to be inferred. The figures represented the ideal rather than

Sir Ernest Moir.

the actual, and should be discounted by anyone who proposed to contract for similar work under similar conditions. He felt that his firm was greatly complimented by what the Author and Mr. Palmer had said as to the plant employed. That used for the entrance-lock, though ideal, would have been extravagant if it had had to be bought for the purpose; but his firm happened to have in stock the goliath cranes which had been used for the construction of the Malta and other breakwaters. He thought the design of the lock was exceedingly bold; at one time he had thought some of the walls were too light, but he had been wrong. The piles for the jetties had been mentioned. The main piles would have had to be cut, and a junction made, if the club-foot precast struts had not been used. The main piles had only to be roughened, and the club-foot was grouted on as the work was carried out. There was sufficient friction between the club-footed diagonal and the pile to take up more than the vertical stresses on the diagonal, and an improved result was obtained. No trouble had been experienced in driving the piles; they were driven about 12–15 feet in gravel, and did not crack anywhere. He was very pleased to see particulars of the dock placed on record, and felt sure there were many matters of interest which the Author might add to the Paper, and which would make it even more valuable. He understood that, in designing lock-gates, the girders which formed the gates were generally treated on a horizontal basis and without any reference to the support which might be derived from the clapping-sill, so that, as the depth increased, they were made either heavier or more frequent. He thought a design might be got out making for a lighter gate, by putting a diagonal girder from the clapping-sill to a few feet below the top of the heel-post, and, through such a system, making use of the clapping-sill for a purpose for which it was generally well designed. It might interest some young engineer to work out in his spare moments the details and weights of a system supported round three sides instead of two. He confessed he had not done it himself, however.

Mr. Du-Plat-Taylor.

Mr. F. M. G. DU-PLAT-TAYLOR remarked that the composition given by the Author for the ballast used showed that it was very similar in nature to dredged Thames ballast, but considerably sandier. The average composition of dredged Thames ballast was shingle 66 per cent. and sand 33 per cent., the weight ranging from 114 to 122 lbs. per cubic foot. He was interested in the matter because, some years ago, he carried out some experiments at Tilbury, which he had recently described,[1] to determine the best ratio of

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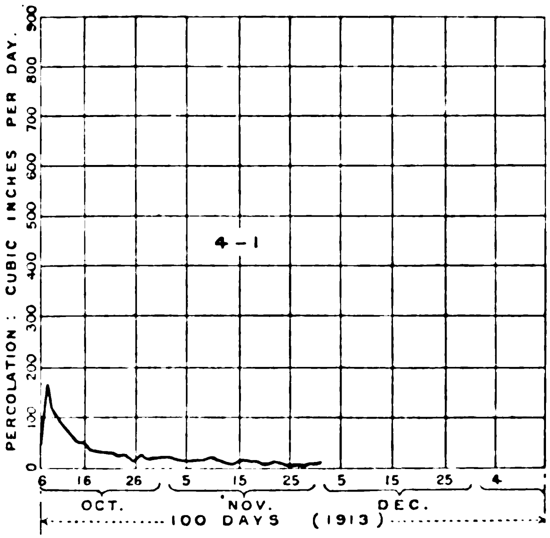
[1] Minutes of Proceedings Inst. C.E., vol. ccxv, p. 172.

Mr. Du-Plat-Taylor. sand to voids. The additional quantity of *sand*, above the voids in the aggregate, which gave the best result at Tilbury was 5 per cent. The Author's figures gave an excess of *mortar*, over the interstices in the aggregate, of 15 per cent. For the 6-to-1 mixture, which was the one tested at Tilbury, the Author's figure of average crushing load at 3 months was 147 tons per square foot, while at Tilbury the figure obtained was 147·6 tons per square foot at 28 days.

The Author. The AUTHOR, in reply, expressed his appreciation of the kind way in which the Paper had been received and the generous remarks that had been made regarding his own association with the work.

Fig. 17.

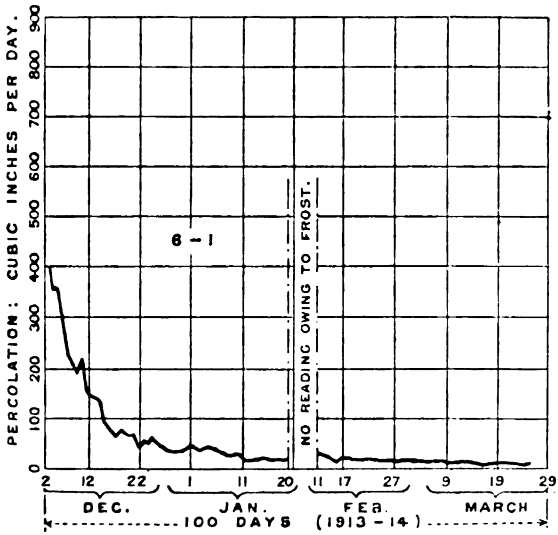


The jetties in the dock were much appreciated by the shipping and lighterage interests, and appeared to be quite successful in their purpose, especially when dealing with import cargoes. Many years must elapse before a definite opinion could be formed of the wisdom of founding the south quay-wall at a higher level, due to the jetties and the sloping bottom. The placing of the vents in the dry dock through the walls just above the bilge altars, instead of the original intention of carrying them through the floor, was due to a happy inspiration on the part of Mr. Palmer, and was quite successful. Several speakers had complained of the shortness of the Paper, and had pointed out that much more information could have been given.

The Author.

It had been desired, however, that the Paper should not exceed the usual limits in length, and it had been curtailed and some of the illustrations omitted, much to his regret. The level of the ballast over the site varied roughly from 10 feet to 20 feet below Ordnance datum. The blocks in the dry dock were kept 1 foot higher than the sill, in order to give comfortable working-space under the ship's bottom. It was not likely that any dock-master would attempt to put into dry dock a ship whose draught gave only 1 foot clearance over the sill. The lower back braces of the jetties had a pre-moulded eyelet at the lower end only, which was slipped over the back piles to the proper position. The upper ends of the pre-moulded members had the steel bars extended for connection to the extension of the middle row of piles in the usual way. The piles were driven into ballast, and there had been practically no trouble due to the shattering of the heads or cracking of the piles. The percolation tests were carried out over a period of about 3 years, and four blocks were kept under test at one time. A timber framework tower was built, 40 feet high, with a tank at the top, fitted with a water-supply pipe and ball cock. Four connections from the tank were carried to the blocks under test at ground-level. The area under pressure in every case was 1 square foot. The blocks were 12-inch cubes, placed in cast-iron boxes open top and bottom, and the sides were filled in with a bituminous

Fig. 18.



The Author. mixture to prevent the water from getting through without passing through the concrete cube. Upper covers were fixed, with a pressure inlet, and the bottom was left open. Blocks were tested both with the brackish water from the dock and with fresh water from the Metropolitan Water Board's mains. There was no observable difference in the percolation or in the rate at which the percolation decreased. On the underside of the blocks, as the percolation proceeded, small stalactites were formed, and these, on analysis, were found to be sodium carbonate (Na_2CO_3, H_2O). It appeared probable that this was the substance which caused the closing of the pores. Typical percolation diagrams for 4-to-1 concrete and for 6-to-1 concrete were shown in *Figs. 17* and *18* (pp. 406 and 407) to the same scale as the 8-to-1 diagram given in the Paper (p. 391). A comparison had been made between the safe load on the ballast (5 tons per square foot) and the breaking strength in compression of 8-to-1 concrete (103 tons per square foot). Notwithstanding the great improvement in the quality of cement in recent years, the Author would not advocate the use of a weaker mixture than 8-to-1 for works of this class.

Correspondence.

Mr. Buckton. Mr. E. J. Buckton remarked that the Paper placed on record, in a clear and concise form, very useful data for the dock- and harbour-engineer. It was difficult to see how the site available could have been used to better advantage. The jetties, although not new in principle, were novel in their extensive application. This method of working ships by cranes on jetties had been in use in isolated cases for many years, and with certain classes of shipping it had proved a success. The result of its adoption on such a large scale would be watched with interest. In the work described, it had been possible to construct a lighter section of quay-wall behind the jetties, thus effecting a considerable saving in the cost of the wall, as a set-off against the cost of the jetties. The ballast formation was good enough to permit of a safe slope of 1 in 5; but, of course, in some materials such a slope would not be possible. It might prove cheaper and better in certain formations to build a deep-water quay-wall along the outer face-line of the jetties, with gaps forming entrances to comparatively shallow barge-areas surrounded by lighter quay-walls. That would permit of the easy filling-in of any of the barge-areas at a later date, should

Mr. Buckton.

changes in shipping methods make it desirable. As usual, bores had been made and trial holes sunk, but nothing was said of any tests of the bearing-pressure of the ground. In this particular case, the foundations being in ballast or chalk, it might have been considered that there was already sufficient local knowledge, and that tests were unnecessary. If, however, tests had been made, it would be instructive to know how they had been carried out and what the results were. No particulars were given of the loads for which the quay-walls were designed, but both the north and south quay-walls appeared to have an excess of concrete at all the steps, or, alternatively, the horizontal sections of the walls at the dock-bottom were too weak. Although an appreciable batter was a disadvantage in working shipping, a slight batter could usually be adopted without causing inconvenience, and, in general, was to be preferred to a vertical face. On any ordinary foundation some movement, however slight, must take place in dock-walls, first due to the general compression of the ground during the construction of the walls, and later due to the compression at the toe caused by the thrust of the filling and surcharge behind the walls. The former was more or less uniform, and usually resulted in a vertical settlement, which was taken up in the construction of the walls; but the latter tended to tilt the walls forward and occurred when a wall was nearly or entirely completed. It was better to have walls with variations in batter than walls leaning forward. Every care seemed to have been taken in the design of the entrance lock and the dry dock. Keeping the sills of the entrance all at one level, although it added to the initial cost, was certainly a great advantage to the dock-maintenance engineer over the usual practice of building the inner sill at a higher level than the outer sill, since the gates were not then interchangeable. Most of the wrought fittings were galvanized. It would be interesting to have evidence of the necessity for galvanizing, when fittings were submerged and never exposed to the air. Two features of interest were the spring spear-post and scrapers of the lock-gates. The Author's opinion as to whether those were likely to prove a success would be welcome. The dry dock was exceptionally well equipped. It was not usually economical, as regarded initial cost or maintenance, to build a very long dry dock to allow for possible future developments in ship-design. The building of a relatively light arched end wall, with the side walls slightly extended to allow of easy lengthening of the dock at a later date, was a wise provision. The general design of the buildings on the north side of the dock was well adapted for dock-buildings, but it was doubtful if the use of the underslung cranes on the first

Mr. Buckton. floor would prove to be the best way of handling goods. The transit-sheds on the south side of the dock appeared to have insufficient headroom to take full advantage of modern stacking-machinery. It was to be regretted that such fine, long, straight quays should have been provided with two lines of rail. If the quays were to be used for railway work at all, three lines were necessary for the efficient handling of the wagons. It was difficult to see, from Fig. 2, Plate 7, how the rails on the south side of the dock could be worked properly.

Dr. Cunningham. Dr. Bryssox Cunningham desired to express his appreciation of the Paper. He asked whether it would be possible to amplify somewhat the simple statement of cost. Despite the fact that the war had interfered materially with the standard of prices for comparative purposes, such information was always serviceable as a general guide, *mutatis mutandis*, and he would suggest that the approximate costs, respectively, of (1) the lock, (2) the dry dock, and (3) the wet dock proper, would enable engineers to form some idea of the financial allocation of the whole outlay. The presence of suitable gravel on the site had enabled a considerable saving to be effected in the cost of the concrete work. In the construction of the south West-India dock, Millwall, where a similar bed of gravel was in evidence, the cost of the concrete was as low as 12*s.* 6*d.* per cubic yard; it would be interesting to know the unit cost of the concrete at the King George V Dock. The dock was a notable undertaking, and not the least interesting feature of the design was the system of jetties on the south side. Perhaps the Author would state how far the arrangement had proved suitable for the conditions it was intended to meet, and whether the quantity of goods deposited directly into barges bore a sufficiently high ratio to that deposited primarily on the quay for sorting (before loading into barges) to justify the special provison made for that method of working. Also, he noted that the width between the jetty and the quay, 32 feet, was considerably more than was requisite to admit a single barge, while it was inadequate to admit two barges of usual width side by side. What was the explanation of that? The quay cranes were of several types, and a comparison of their working-efficiency or, at any rate, a statement of their merits under similar conditions, would be instructive. Was the disposition of six cranes per jetty a satisfactory arrangement, and, if so, in what proportion were the cranes allocated to the hatchways of vessels at the berths? Assuming, say, four holds to be working simultaneously, there would be an excess of quay cranes available per berth, unless two cranes were worked at a single hold, in which case the number would be insufficient. To what extent did the use of ships' appliances supersede the employment of the cranes? The subject of cargo-handling was complex, and it was not altogether easy to put questions which, without causing a good deal of trouble in reply, would enable some general idea to be formed of the precise conditions under which the berths were worked, and the suitability of the arrangement to meet those conditions. The Author might, perhaps, in a few words, give his experiences of the method of dealing with traffic conditions at the new dock. The construction of the dock was a notable achievement, and the solidity and finish of the work was such as to command general admiration. At one time he had been a whole-hearted advocate of strength and durability; but of late years, in view of the transitory character of most enterprises of the kind, and their rapid obsolescence, he could not help feeling that it was possible to execute such works too solidly. He had recently had an opportunity of inspecting piers and jetties of quite light construction at American ports, some of them piled structures of a very flexible character, which accommodated ships of tonnage equal to that of any vessel which entered an English port. The accommodation was obviously provided at far less outlay, and the important consideration was that it served its purpose. Moreover, the future renewal of such structures, called for by rapid developments in the size of vessels, did not entail so heavy an expenditure as would be the case if it became a question of demolishing half a million cubic yards of concrete and a quarter of a million cubic feet of solid granite. A number of years ago he had some experience in the carrying out of a scheme of dock-improvements at the port of Liverpool, which involved cutting through a series of obsolete graving-docks; and the time, labour, and outlay involved in merely destructive operations had greatly impressed him. The question, therefore, was one which should receive the careful consideration of dock-engineers; and, while he did not by any means intend to imply that it had not received every consideration in the present instance, the reflection inevitably obtruded itself during a perusal of the Paper. Dr. Cunningham.

Mr. F. W. DUCKHAM considered that the Paper formed an excellent and well-balanced record, which should be useful for future reference. Procedure had been smooth and had followed the programme except for interruption by the war. Thus there had been no failures such as might point the way to improvements. From his point of view, as agent for construction during the first 6 years, there were a few matters which might be amplified. It was not quite correct that electric power had not been used for cranes. Two jib cranes for Mr. Duckham.

Mr. Duckham. excavation, specially built by Messrs. Booth with electric drive, showed considerable saving in working-costs, in comparison with an otherwise identical pair driven by steam and used on the same work. The chief novelty in the contractors' plant was a pair of concreting machines of a universal character, built on the site at the instance of Sir Ernest Moir. Each was fitted with a large Ransome mixer, served by two winches, one of which excavated the ballast, and the other distributed the concrete by means of a Stoney skip. Those machines also were entirely electric and were most efficient in all respects. By improvements in constructional methods the whole of the open excavation had been done in the dry; the seven inside jetties had been modified in reinforced concrete with a great saving in cost; and the dry dock had been completed by a new method which saved much time and money. The broad and open-minded attitude of the Engineers to the Authority, in accepting such innovations at the instance of the contractors provided an exceptional instance of mutual efficiency.

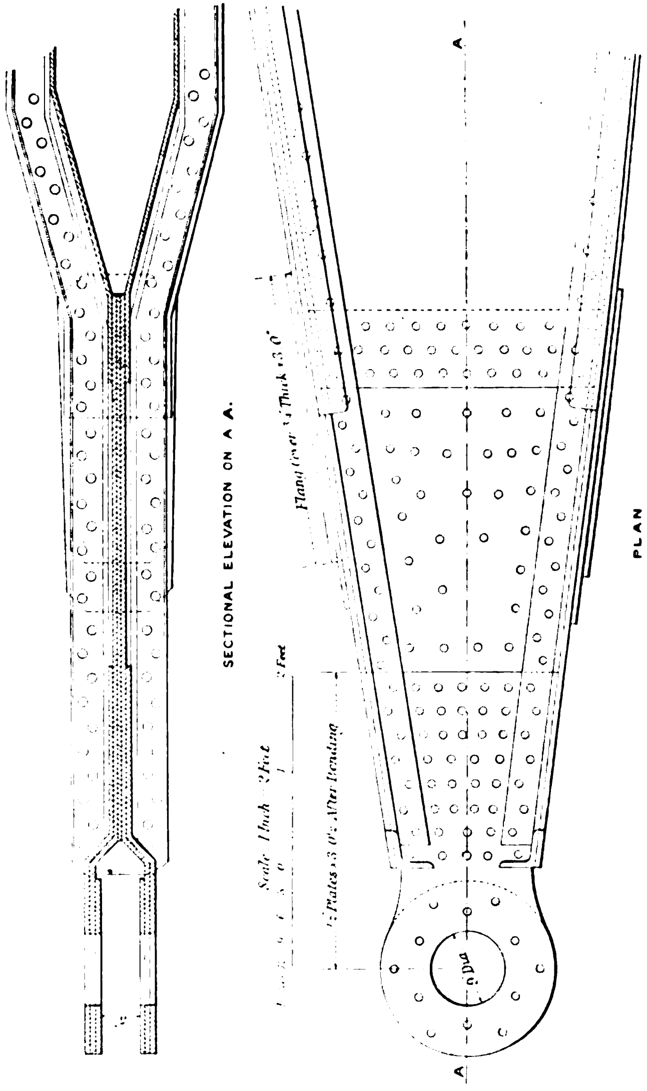
Sir Cyril Kirkpatrick. Sir CYRIL KIRKPATRICK remarked that to him it was a matter of regret that, being in Sweden at the time, he had been unable to take part in the discussion upon the King George V Dock. He desired to make special reference to the great part taken by Mr. Palmer in the design and construction of the work. The dimensions of the dock and the design were his conception, and he was the responsible engineer for 6 out of the 9 years during which the work was in hand, Sir Cyril Kirkpatrick acting as second in command during that period and only taking full control from 1918. At that date the Authority made an amicable arrangement with the contractors to finish the work by departmental labour, owing to difficulties arising out of the war, which prevented the contractors from obtaining the necessary labour. The Author, as Resident Engineer throughout, had shown himself equally capable and resourceful in supervising the work carried out by the contractors and in carrying out the work departmentally.

In the execution of any great work experience was gained for future operations; and he proposed, therefore, to describe how certain difficulties had been surmounted. All who were responsible for executing work during the war realized that many unforeseen contingencies arose, and that unless adequate "priority" was obtained, progress could not be kept up. Thus all steel construction was brought to a standstill, and the lock-gates, having been put on one side for more urgent Government work, were not available when required. In 1918, however, the Government came to the conclusion that the new dry dock would be a great asset, and an

Sir Cyril Kirkpatrick.

acceleration order was therefore given, with high "priority" for all the necessary steelwork. One of the conditions made was that the dry dock should be available for use at the earliest moment possible, before the entrance and lock-gates were ready to take shipping from the river. Vessels making use of the new dry dock would thus enter the Albert dock and pass to the new dry dock by means of the passage between the old and new docks. This course necessitated the immediate completion of the inner pair of lock-gates, and the use of them as a dam against the full head of water until the completion of the middle and outer gates enabled the lock to be flooded. That was a risk which the exigencies of the war rendered imperative, and, had not the armistice taken place, it would have been necessary to take very special precautions to prevent any vessel from accidentally fouling the dam. The armistice, however, took place in November, 1918, and the flooding of the dock was not commenced till January, 1920. It was thus possible to proceed slowly until the second pair of lock-gates were completed, and to step the water down in the inner lock-chamber, thus avoiding the use of a single pair of gates as a dam against the full head. In the flooding of the works in two cases some anxiety was caused. In the first instance a small temporary earthen dam formed of ballast in the lock-chamber, with a slope of 1½ to 1 when dry, flattened out to a slope of 6 to 1 when wet. In the second instance the earth dam in the passage between the old and the new dock had to be fortified by making a toe of heavy rubble to prevent it from flattening out as the water was admitted to the new dock. A point of great interest arose in connection with the construction of the pumping-station for the dry dock (Fig. 5, Plate 7), the floor being fixed at a level of 22 feet 6 inches below dock water-level. In view of the fact that the motors were designed for a pressure of 6,600 volts, it was with some anxiety that the amount of moisture percolating through the walls was observed as the dock was flooded, and it was hoped that it would take up automatically in the course of a few weeks. All endeavours to dry the atmosphere by ventilating fans and heating failed to procure the desired condition, but the difficulty was finally conquered entirely by the building of a 4½-inch brick wall inside, with a cavity behind, 3 inches wide, drained into the sump. A difficulty arose, about 18 months after the opening of the dock, at the lock-gates, which required careful investigation. The design adopted for connecting the crocodile lever was shown in *Figs. 19* (p. 414). It was understood that the late Sir John Wolfe Barry had laid down that gates for an 80-foot entrance should take 1½ minute to operate. Tests showed that the King George V

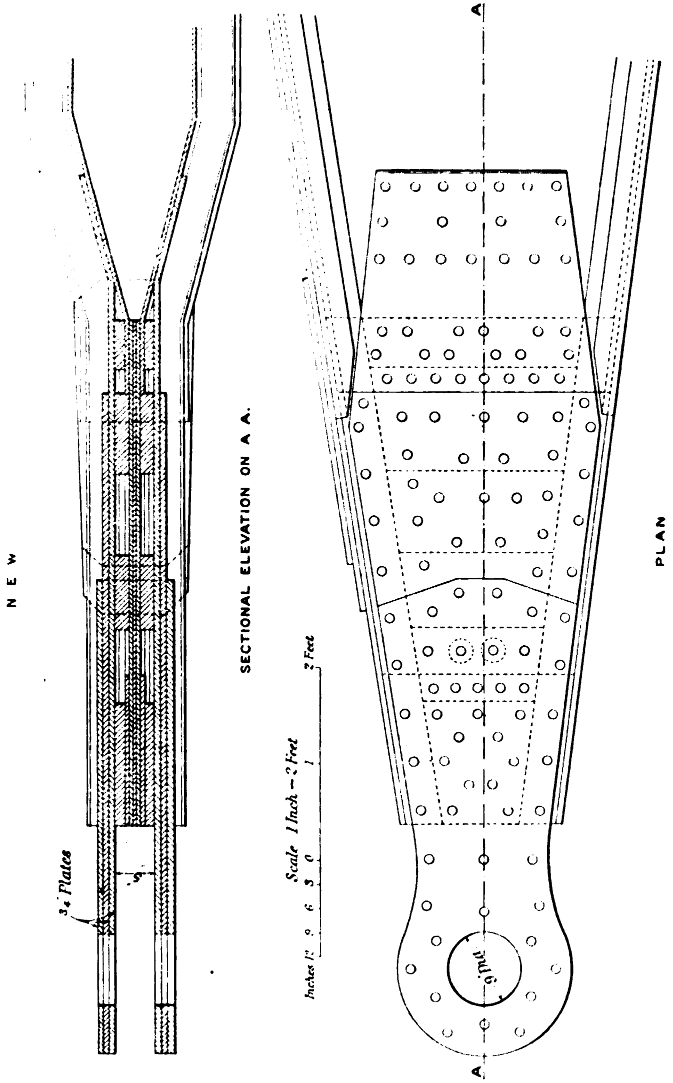
Sir Cyril Kirkpatrick. gates for a 100-foot entrance were operated in a minimum time of 1 minute 12 seconds. It was clear, however, that that high speed was not the cause of trouble. In the course of a few months several of the opening levers developed either a fracture or a crack at the cranked plate near the heel-post. Investigations showed clearly


SECTIONAL ELEVATION ON A A.
PLAN
A
A
Flange Cover ½ Thick × 3.0"
Scale 1 Inch = 2 Feet
1
2 Feet
½ Plates × 3.0" After Bending
2" Dia

Figs. 19.

Sir Cyril Kirkpatrick.

that the cranked plate was most undesirable, and strengthening had therefore been effected, as shown in *Figs. 20.*


NEW
3/4" Plates
SECTIONAL ELEVATION ON A A.
Scale 1 Inch = 2 Feet
Inches 12 9 6 3 0 1 2 Feet
A
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9in
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Figs. 20.

The single-storey sheds on the south side of the dock were built on concrete piles about 37 feet long. The material driven through was 15 feet of newly-tipped excavation from the dock, including

Sir Cyril Kirkpatrick. the upper 8 feet of ballast, 18 feet of peat and clay, and 4 feet into solid ballast. The framework of the sheds was erected in December, 1915, and the level of the columns on the piles was duly checked, but nothing more was done to them until the Government acceleration order enabled progress to be made in August, 1918. It was therefore a surprise, when work was recommenced, to find that the piles had sunk by 1 to 4 inches in the period; and the only possible conclusion was that the settlement of the newly-excavated material had forced the piles downwards. He desired to add his testimony to the able manner in which the work had been carried out by the contractors and the excellence of the plant used by them.

Mr. Reid. Mr. H. CARTWRIGHT REID remarked that the Author described very succinctly works which might be considered one of the finest commercial dock-projects of modern times. The Port of London Authority was to be congratulated on making such an important addition to the premier port of the world. From the autumn of 1918, when facilities were given by the Government for completing the work, to the 21st February, 1921, when he accompanied the first vessel into the new dock, it had been his duty to make periodical visits to the works. To those familiar with the progress of large engineering works during that critical time it would be well known that the difficulties had placed a great strain upon the engineers in charge; and the Author and those associated with him might well feel proud of completing that great work in a satisfactory manner. He had found little to criticize and much to admire in the way the work was being done. When the progress of the work was almost suspended by the war, the Port Authorities had spent a large sum towards the construction of the works and had no immediate prospect of remuneration in the way of additional port-dues to meet the cost of interest during construction. Had they postponed the completion of the work until war prices had ceased, they would probably have found then that the interest during construction equalled the extra cost of finishing the work; and though the new dock had been very expensive, due to these untoward circumstances, the Port Authority, with the primary object of additional port-facilities in view, had adopted the better course of pressing the completion of the works at the earliest possible date. The Author did not attempt to give the probable extra cost due to the war, but it was evident that that must be considerable. It would be interesting, however, if he could state the approximate value per berth of the fourteen berths which had been provided. Omitting the cost of the dock and other items which were to be deducted from the total cost, the value per berth would, Mr. Reid presumed, be about £250,000. Only a premier port like London was likely to make sufficient profit from port- and landing-dues to pay the interest on so large a figure. Could the Author give any figures of the probable earnings of those berths, after deducting the expense of working them? The most striking feature of the design was the arrangement of the jetties on the south side of the dock. That bold attempt to meet the special condition prevailing in the Thames of a large proportion of unloading into barges would be watched with interest. If the Author could give an approximate idea of the relative values, per linear foot, of the berths on the south side compared with those on the north side, that information would be valuable. Were similar rates charged to ships for the berths on both sides, and had shipping firms had sufficient experience to indicate any preference for one type of berth? The water-level of the new dock, which had direct communication with the existing system of docks, was maintained at the same level of 2 feet 6 inches above Trinity high water. The maintenance of that high level, no doubt, simplified the locking arrangements and prevented any silting due to river-water entering the dock, but the cost of pumping must be considerable, in view of the large area of the dock and the dimensions of the lock. Figures showing the annual cost of pumping would be very instructive. **Mr. Reid.**

The AUTHOR, in reply, observed that he had noted with interest Mr. Buckton's suggested alternative for the jetties in the dock. It was doubtful, however, if the system proposed would effect any substantial saving in initial cost; and, although the Author did not regard the filling-in of the bargeways as a likely contingency, the bridging of the gap by a reinforced-concrete structure similar to the jetties would present no difficulty. No tests had been carried out to ascertain the bearing pressure on the ground at the formation-level of the quay-walls. The calculated intensity of pressure at the toe of the walls was 5 tons per square foot, and, the level of the foundations being in all cases in ballast or chalk, tests had been considered unnecessary. The quay-walls were designed to carry a surcharge of 3 tons per square yard. The vertical face was adopted for convenience in working shipping and simplicity in construction, and there had been no tilting forward. The spring spear-posts and scrapers of the lock-gates so far had proved satisfactory. **The Author.**

The underslung cranes on the first floor of the north quay-sheds were at present an unqualified success, being especially useful in stacking bales of wool and hogsheads of tobacco for storage purposes. The sheds on the south side of the dock were seldom used for storage

The Author. over long periods, and the necessity for using stacking-machinery did not frequently arise. Such piling as might be required was carried out by means of 15-cwt. runabout electric cranes, which could be easily worked in the sheds, and had proved adequate for the requirements. The two lines of rails had been installed as an additional facility in cases of emergency, and the double crossovers at the end of each shed had enabled the necessary traffic to be worked satisfactorily. Practically the whole of the rail traffic was dealt with at the rear of the sheds, where three lines of rails had been provided, with all the necessary crossovers and connections.

As stated by Dr. Cunningham, the war had interfered materially with the standard of prices for comparative purposes. It could not be stated, in respect of any particular section of the work, that the expenditure was wholly pre- or post-war. The cost covered a period when substantial variations in prices of labour and materials were of frequent occurrence. It was therefore practically impossible to reduce the costs to a pre-war basis. For this reason details were excluded from the Paper, and would, if given now, be more misleading than useful. Owing to the presence of suitable gravel on the site the cost of concrete had been very low. The prices had ranged, at pre-war rates, from 8*s*. 4*d*. per cubic yard for 8-to-1 mass work in the dock, and 9*s*. 9*d*. per cubic yard for 6-to-1 work in the lock, to 17*s*. 2*d*. per cubic yard for the 6-inch facing of 4-to-1 quality. The prices quoted included shuttering.

The experience gained in connection with the jetties in the dock had not been sufficient to enable complete and satisfactory data to be furnished with regard to their successful use. The working of ships' cargoes in the King George V Dock was entirely in the hands of the various shipping-companies. Naturally, practice varied considerably, not only as between the different companies, but in relation to the character of the cargoes. Evidence had been taken however, from more than one source, and it had been found that often upwards of 80 per cent. of a ship's cargo was removed by barge. No records were available as to the actual tonnage discharged direct into craft, but it might be borne in mind, in this connection, that the arrangement of the jetties provided additional facilities for loading into barge either direct or after sorting and marking on the quay, and it might be taken generally that there was a substantial saving in time in discharging a vessel at the south quay compared with the north quay. The width between the jetties and the south quay had been fixed at 32 feet to allow for possible development in the size of craft, and to bring the face of the jetties clear of the 5-to-1 slope at the dock-bottom. It had been borne in mind, in fixing this dimension, that barges were then being designed 24 feet wide. The Author.

The quay cranes, although comprising several different types, were all fulfilling satisfactorily the requirements of the Port Authority, but no figures were available as to efficiency. The object of the disposition of the six quay cranes per jetty was to give the maximum facility possible without causing congestion. In practically every case the full complement of cranes was brought simultaneously into use. Sometimes the cranes each worked at separate hatchways, and more generally they worked in two pairs, and two singly. For quayside working the cranes practically superseded the use of ships' appliances, these being used only for "overside" work into barges on the dock side of the ship.

The Author appreciated Dr. Cunningham's remarks as to the possible tendency to execute works of this character too solidly, and agreed that the question was one requiring careful attention. At the same time, and while he realized that the suggestion made was general and not specific in character, he deprecated the adoption of any lighter form of construction under conditions similar to those under consideration. In contrast to the American piers and jetties instanced, dock-walls not only functioned primarily as shipping-berths, but in the present instance they also formed a dam to retain water at a level approximately 10 feet above the surrounding industrial districts of North Woolwich and Silvertown.

As to the probable extra cost of the works due to the war, for reasons previously stated it was practically impossible to reduce the actual costs to a pre-war basis, but so far as could be calculated, the additional cost attributable to this cause was approximately £2,000,000. The value per berth of the fourteen berths provided, including roads, railways, sheds, cranes, and equipment, together with the whole charge of the entrance-lock and passage, was approximately £275,000. All these berths had been allocated to shipping-companies, and the direct earnings were by way of rent-charge calculated on an area basis. It was regretted that the amount of these charges and other earnings per berth could not be given. The approximate cost per linear foot of the south and north quay-walls, including trench-excavation, but excluding quay-paving, sheds, railways, etc., was £12 10*s* and £20 respectively, at pre-war rates. The jetties in the dock cost £40 per lineal foot at post-war rates, representing approximately twice pre-war values.

All the berths were allocated to shipping-companies before or soon after the opening of the dock. The question of preference

The Author. as between the north and south sides, based on general experience, had therefore not arisen. As already indicated, however, there was every reason to believe that the experiment of providing jetties to the southern berths had proved successful.

As suggested by Mr. Reid, the cost of pumping to maintain a water-level in the dock of 2 feet 6 inches above Trinity high water was considerable. The total area of water impounded in the King George V Dock and the Victoria and Albert Docks was 251 acres; and the working-cost of pumping, exclusive of depreciation and interest on capital, was at present about £12,000 per annum.

(*Paper No. 4219.*)

"Railway Economics for Extra-European Railways."

(*Paper No. 4289.*)

"Reinforced Concrete on the Chinese Railways."

By HAROLD STRINGER, B.A., Assoc. M. Inst. C.E.

Author's Reply to the Correspondence.[1]

The AUTHOR, in reply, remarked that the gradient of 1 in 23½ over the Kicking Horse Pass on the Canadian Pacific Railway had been cut out, the ruling gradient there being now 2·2 per cent. This regrading was the heaviest piece of work on the line, there being two spiral tunnels and a 5-mile summit tunnel, besides some heavy bridgework on this length. The radius of curvature over this length was 500 feet; rail-wear and creep troubles appeared to be serious. Mr. Money's surmise as to an error in stating rail-length in America was correct. This should be 33 feet, as stated by him. The Author.

In reply to Mr. F. R. Johnson, it was stated below Table VI (p. 307) that the slabs suggested there were designed for 85-lb. rail loading. More recent regulations had adopted Cooper's E.50 loading as standard for the Peking–Mukden Railway. The rail reinforcement was placed head downwards, as previous experience with steel joists indicated that the protective coat of concrete was liable to spall off under traffic.

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[1] The Papers, with the Discussion and Correspondence upon them, were printed in vol. ccxv.—SEC. INST. C.E.

SPECIAL GENERAL MEETING.

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17 April, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The following Notice convening the Meeting was read :—

THE INSTITUTION OF CIVIL ENGINEERS,

Great George Street, Westminster, S.W. 1.

9th December, 1922.

" Dear Sir,—In accordance with the requirements of the Supplemental Charter of the 24th February, 1922, notice is hereby given that a Special General Meeting will be held at the Institution on Tuesday, the 17th April, 1923, at 6.0 p.m., when the following Resolution will be submitted :—

> "That the revised By-laws as approved by the Council on the 5th December, 1922, for submission to the Corporate Members and now submitted to this Meeting be approved and adopted, in lieu of and in substitution for the existing By-laws, and that the Council be authorized to submit such revised By-laws to His Majesty's Privy Council for approval in accordance with the provisions of Clause 3 of the Supplemental Charter dated the 24th February, 1922.

" If you desire to vote at the above Meeting without attending personally, will you please sign the enclosed voting paper and dispatch the same so as to reach the Secretary not later than the 12th day of April, 1923."

Yours faithfully,
H. H. Jeffcott,
Secretary.

The Chairman informed the Meeting that 1,994 votes in favour of the Resolution and 30 votes against it had been registered by means of voting papers.

The Chairman reminded the Meeting that any voting papers sent in by members present at the Meeting could with his sanction be withdrawn, so as to facilitate voting by a show of hands at the Meeting, and that it had been suggested that this should be done. Accordingly he put the suggestion to the Meeting and it was unanimously agreed to.

The Chairman then explained the steps taken by the Council to obtain suggestions from members as to alterations in the By-laws, and stated that the revised By-laws as submitted to the members were the result of the Council's very careful consideration of all these suggestions.

The Chairman then moved, and Sir Charles Morgan, Vice-President, seconded the following Resolution :—

> That the revised By-laws as approved by the Council on the 5th December, 1922, for submission to the Corporate Members and now submitted to this Meeting be approved and adopted, in lieu of and in substitution for the existing By-laws, and that the Council be authorized to submit such revised By-laws to His Majesty's Privy Council for approval in accordance with the provisions of Clause 3 of the Supplemental Charter dated the 24th February, 1922.

An amendment was thereupon moved by Mr. J. D. C. Couper, Member, and seconded by Mr. J. Mitchell Moncrieff, Member :—

> That the Meeting be adjourned until 6 p.m. on the 8th January, 1924.

After considerable discussion the amendment was put to the Meeting, voted on by a show of hands, and was declared by the Chairman to be lost.

The following amendment was then moved by Mr. Frank Turner, Associate Member :—

> That in Section VIII, Constitution of the Council, Paragraph 1 of the Revised By-laws, as approved by the Council on the 5th December, 1922, and now submitted to this Meeting be altered as follows :—
>
> "The number of Members elected to the Council in any year shall be thirty-four, inclusive of the President and Vice-Presidents, but exclusive of such of the Past-Presidents, if any, as may be appointed by the Council", and to add, as a final sentence :
>
> "This number shall include at least three Associate Members."

This amendment was not seconded, and after some further discussion was declared by the Chairman to be out of order.

The original Resolution was then put to the Meeting and voted on by a show of hands, the result of which was :—For, 78; Against, 12.

In addition the postal votes were :—For, 1,959; Against, 28.

The Chairman declared the Resolution carried.

The Meeting then ended.

ANNUAL GENERAL MEETING.

24 April, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

The Notice convening the Meeting was taken as read, as well as the minutes of the Annual General Meeting of the 25th April, 1922, which the President was authorized to sign.

The following Report of the Council upon the Proceedings of the Institution during the Session 1922–1923 was read, the statement of Accounts being taken as read.

After consideration, it was resolved,—That the Report of the Council be received and approved, and that it be printed in the Minutes of Proceedings.

A statement (Appendix II, p. 454) of the financial position of the Institution, with special reference to the question of the revision of subscription rates, was read.

After consideration, it was, on the motion of the President, seconded by Sir Charles Morgan, Vice-President, resolved :—

That the statement be approved and adopted.

The Scrutineers reported the election of the Council for 1923–1924, as follows[1] :—

President.

Sir CHARLES LANGBRIDGE MORGAN, C.B.E.

Vice-Presidents.

Basil Mott, C.B.
Sir William Henry Ellis, G.B.E., D.Eng.
Frederick Palmer, C.I.E.
Sir Archibald Denny, Bart.

Other Members of Council.

Sir John Cadman, K.C.M.G., D.Sc.
Charles Claude Carpenter, C.B.E., D.Sc.
George Muirhead Clark, M.A.
Peter Chalmers Cowan, D.Sc.
Rookes Evelyn Bell Crompton, C.B., *Col.* R.E. (T.).
William Wylie Grierson, C.B.E.
Sir Robert Abbott Hadfield, Bart., D.Sc., D.Met., F.R.S.
Kenneth Phipson Hawksley.
Sir Brodie Haldane Henderson, K.C.M.G., C.B.
Ernest Prescot Hill.
George William Humphreys, C.B.E.
Sir Cyril Reginald Sutton Kirkpatrick.
Sir Murdoch Macdonald, K.C.M.G., C.B., M.P.
John MacGlashan.

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[1] The Council commence their year of office on the first Tuesday in November, 1923.

Joseph Primo Maxwell.

Sir Henry Percy Maybury, K.C.M.G., C.B.

Sir John Monash, G.C.M.G., K.C.B., D.C.L., LL.D., D.Eng.

John Bonsall Porter, Ph.D., D.Sc.

Sir Hugh Reid, Bart., C.B.E., LL.D.

George Richards, F.C.H.

Matthew Henry Phineas Riall Sankey, C.B., C.B.E., *Capt.* R.E. (*ret.*).

Sir John Francis Cleverton Snell.

William Archer Porter Tait.

Ernest Frederic Crosbie Trench, C.B.E., M.A.

John Duncan Watson.

Sir Alfred Fernandez Yarrow, Bart., F.R.S.

Resolved,—That the thanks of the Meeting be given to the Scrutineers, and that the Ballot-Papers be destroyed.

Mr. E. R. Dolby responded on behalf of the Scrutineers.

Resolved,—That the thanks of the Institution be given to Messrs. P. D. Griffiths and M. F.-G. Wilson, Auditors, and that they be reappointed Auditors for the current financial year.

Mr. Wilson acknowledged the resolution.

Resolved,—That the thanks of this Meeting be accorded to Dr. W. H. Maw, President, for his conduct of the business as Chairman of the Meeting.

The President acknowledged the Resolution.

The proceedings then ended.

REPORT OF THE COUNCIL, 1922–1923.

In presenting the following report upon the state of the Institution, the Council have to record certain important steps which they have taken during the past year by which they have sought to develop and increase the service which the Institution is ever endeavouring to render to the Civil Engineering profession, particularly in the direction of the advancement of engineering science, with which the interests of its members and students are so closely identified.

By-Laws. In the first place, the revision of the By-Laws, to which reference was made in the last Report, having been completed, the amended draft was submitted to the members at home and abroad in order that an opportunity should be given them to record their votes for or against the proposed alterations. In the result, the revised By-Laws have been adopted at a Special General Meeting held on the 17th April, a very large proportion of the votes received being favourable to the amendments made. The By-Laws thus revised are being submitted to the Lords of the Privy Council in order that their approval may be obtained to the revision, in accordance with the provisions of the Supplemental Charter granted to The Institution in February, 1922.

Publications. The Council have recently given very careful consideration to the whole subject of the Institution publications, and they have decided upon certain changes, which will be referred to in another part of this Report, and which, it is believed, will enable the funds available for such purposes to be expended to the best advantage so as to increase the usefulness of the publications to the members.

Heat-Engine and Boiler Trials. Having regard to the developments which have occurred in the methods of engine- and boiler-testing since the Report of the Institution Committee on Steam-Engine and Boiler Trials was published in 1913, the Council have resolved to revise and extend the Report. A new Committee has been appointed, composed of members nominated on the invitation of the Council of the Institution by a number of the principal engineering and scientific institutions in the country, and certain other societies have also accepted invitations to co-operate in the work. In addition to the revision of the old Report, sections dealing with the testing of Steam Turbines for Land and for Marine Purposes, Turbo-Alternators, Gas Engines, Petrol and Paraffin Engines and Heavy Oil Engines will be incorporated in the new Report.

The future development of the Local Associations of Corporate Members and Students of the Institution is receiving the very close consideration of the Council. Further reference will be made in this Report to the activities of these Associations, but it may be mentioned here that the questions which have engaged the Council's attention include the basis upon which grants are made to the Associations, and the delimitation of areas. Local Associations.

In deliberating on these developments, as well as on other measures such as the provision made for a lending library and for additional and Informal Meetings, the Council have had constantly to bear in mind the present financial position of the Institution, with particular regard to the fact that the question of subscriptions is being reviewed. Whilst the strictest economy has been practised, it will be realized that printing is still costly, that certain other charges can only be reduced very gradually, and that over the large part of the expenditure due to rating and taxation the Council have no control. Under these circumstances, the Council are of the opinion that the reductions which have been effected had better be expended in developing the services of the Institution to its members in the manner described above. Library.

Referring next to matters of routine, the Council have to report that whilst there is a steady increase in the roll of the Institution, the general conditions of trade and industry resulting from the war continue to affect adversely the rate of such growth when comparison is made with the years immediately preceding the war. On the other hand, the number of Students admitted into the Institution shows a considerable increase, and is, indeed, not far short of the best recorded in any previous year. This is partly due to the less exacting requirements in the matter of specific arrangements for practical training which were introduced in 1920 as a temporary measure to help college students who found difficulty in making the necessary arrangements before they had completed their college courses. The high standard of educational qualifications required has, however, been maintained, and the Council are of opinion that the growth of the Student class under these circumstances is a most encouraging sign, particularly as this class always provides a large percentage of the candidates elected to Associate Membership. **Elections and Admissions.**

The Council have investigated all alleged breaches of the regulations governing the professional conduct of the members which have been brought to their notice during the current Session, and have taken disciplinary action where it appeared to them to be necessary in the manner prescribed in the By-Laws. **Professional Conduct.**

Scholarships.

The Council wish to take this opportunity to remind the members and Students of the scholarships which are administered by the Institution. These consist of the Yarrow Scholarships and the Lindley Scholarship, to enable engineering students to complete their training, and of the Palmer Scholarship, the object of which is to assist the sons of civil engineers to graduate (not necessarily in mechanical science) at Cambridge University. It is proposed in future, when funds are available, to consider applications for all scholarships twice yearly, namely, in the spring and in the autumn; and the Council hope that there will be no lack of suitable candidates.

Yarrow Scholarships have been awarded during the past year to Graham Ashworth, Reginald William Mountain and David Stevenson Watt, the two first-named being Students of the Institution, to enable these scholars to complete their engineering training.

Hawksley Prize.

It may be well to recall that the object of the Charles Hawksley Prize is to encourage among young engineers the æsthetic treatment of engineering structures, it being required that the designs submitted in competition for it should combine artistic merit with excellence in constructional design. Eighteen Associate Members and Students entered for this year's competition, and the Council, having considered the report of the judges who examined the designs, have come to the conclusion that the standard reached does not justify any award of the Prize. They have decided, however, that, having regard to the good draughtsmanship and evident care displayed by two of the candidates, namely:—Frank Whyte, Associate Member, and Frank William Edward Tydeman, Student, these competitors shall, notwithstanding defects in their designs as a whole, be honourably mentioned, and that a sum of £20 be divided equally between them.

The results of the competition are such as to show that the subject of the Prize as mentioned above is not receiving from young engineers the consideration which it deserves, and the Council hope that these competitions may in future have the effect of causing greater attention to be given to the matter.

Associate Members Committee.

The Council have given careful consideration to the proposal, made at the last Annual General Meeting, as to the formation of an Associate Members Committee to report on matters affecting the interests of Associate Members. In the result, they came to the conclusion that it would not be expedient to take such action.

Local Associations.

The Council have to report that the Birmingham, Manchester, Newcastle-upon-Tyne and Yorkshire Associations of Corporate Members and Students continue to make satisfactory progress, and that their meetings, etc., during the current Session have been well attended. The Council have agreed to the formation of a similar branch at Bristol (to replace the Bristol, West of England and South Wales Association of Students), to be named the Bristol, Cardiff and Swansea Association of Corporate Members and Students. They wish it equal success to that which has attended the other branches referred to above.

Having regard to the future development which may be anticipated in connection with these local branches of the Institution, it is not out of place to recall the objects for which they have been formed. These may briefly be said to be to provide (originally for Students, and now for both members and Students) a centre where Papers may be read and discussed, and where the fostering of good feeling and the promotion of a corporate spirit amongst the members in a particular district may be encouraged, thus providing provincial members with some of the advantages enjoyed by the London members.

The Council, in considering the yearly grants which are made to the Local Associations, have come to the conclusion that these should be awarded on a basis depending primarily on the number of Corporate Members and Students who take part in their activities. This necessitates the delimitation of the Local Association areas, and these areas have been determined by a 20-miles radius from the centres of the Associations. This will not, however, preclude a member or Student residing outside the 20-miles radius, who actively supports an Association and attends its meetings, from being included on the roll of that Association.

Appointments and Nominations.

The following nominations and appointments have been made or confirmed by the Council during the past year :—

Main Committee of the British Engineering Standards Association.	Sir John Aspinall, Dr. Charles C. Carpenter, Sir John Dewrance, Sir William H. Ellis, Sir Maurice Fitzmaurice, Dr. W. C. Unwin, Mr. M. F.-G. Wilson.
General Board of the National Physical Laboratory.	Sir Robert Elliott-Cooper, Dr. W. C. Unwin.
Governing Body of the E. D. Denning Trust.	Mr. E. P. Hill.
Court of the University of Bristol.	Mr. W. W. Grierson.
Governing Body of the School of Metalliferous Mining (Cornwall).	Mr. J. H. Johns.

Governing Body of the Imperial College of Science and Technology.	Dr. W. C. Unwin.
L.C.C. Tribunal of Appeal (General Powers Act, 1909).	Sir Robert Elliott-Cooper.
Advisory Panel on Transport (Ministry of Transport).	Sir John Aspinall, Mr. John A. Brodie, Mr. O. H. R. Bury, Sir Maurice Fitzmaurice, Sir John P. Griffith, Mr. T. R. Johnson, Mr. J. A. Saner.
Court of the University of Liverpool.	Mr. John A. Brodie.
Royal Engineer Board.	Sir Brodie H. Henderson.
Selection Committee for the appointment of Assistant Executive Engineers in the Indian Service of Engineers and State Railways Departments.	Sir Robert Elliott-Cooper.
Court of Governors of the University of Sheffield.	Sir William H. Ellis.
British Empire Exhibition, 1924.	Col. R. E. B. Crompton, Mr. E. W. Monkhouse, Capt. H. Riall Sankey.
Committee on Fourth International Road Congress (1923), Seville.	Mr. G. W. Humphreys.
Committee on Standard Tests for Water Turbines.	Professor S. M. Dixon, Mr. J. S. Highfield, Mr. Evan Parry.
Engineering Joint Council.	Sir Maurice Fitzmaurice and Mr. W. B. Worthington—for Session 1922–23. Sir Maurice Fitzmaurice and Sir Charles L. Morgan—for Session 1923–24.
International Air Congress.	Sir Henry Fowler.
British Electrical and Allied Manufacturers' Association (World Power Conference, 1924).	Col. R. E. B. Crompton.
The Institution of Mechanical Engineers (Alloys and Iron Research Committee).	Sir Robert Hadfield, Bart.

Sea Action.

The Committee on the Deterioration of Structures Exposed to Sea Action, under the Chairmanship of Mr. M. F.-G. Wilson, have carried on the work commenced in 1916.

During the past year reports have been received on the specimens of steel and other metals which had been exposed for corrosion tests at Auckland, Colombo, Halifax, and Plymouth.

A number of specimens of Baltic and Swedish fir, impregnated with creosote and certain poisons, as well as untreated control specimens, have been exposed at Lowestoft. It is too early to speak with certainty as to the preservative value of these poisons, but so far the results appear to be promising, and it has been decided to test some of the poisonous substances at Colombo, where the action of sea worms is extremely active, by exposing specimens injected with creosote in which these poisonous substances have been dissolved.

The mechanical tests of timbers mentioned in the Council's previous report are being continued by Professor Dixon, at his laboratory at the City and Guilds (Engineering) College, and valuable information has been obtained as to the comparative strengths of creosoted and uncreosoted timbers.

The Committee are now undertaking a comprehensive series of tests as to the preservation of steel by different paints and preservatives. A number of steel plates have been prepared from a single ingot, and these, after being coated with paints selected by the Committee, are being exposed by Mr. F. E. Wentworth-Sheilds at Southampton.

The Third (Interim) Report of the work carried out by the Committee during the year ending the 31st March, 1922, is now in the press and will shortly be ready for issue.

International Navigation Congress.

During the coming summer, the Institution will have the pleasure of placing its rooms at the service of the Permanent International Association of Navigation Congresses, which is holding its thirteenth Congress in London under the Chairmanship of Lord Desborough. The Congress will be opened on Monday, the 2nd July, and all arrangements are in the hands of an organizing Committee, of which Mr. R. B. Dunwoody is the Honorary Secretary.

Co-operation of Engineering Societies.

The Council had occasion in their last Report to refer briefly to the preliminary steps which had been taken to obtain the closer co-operation of engineering societies. The Joint Committee having reported in favour of such co-operation, the Engineering Joint Council, consisting of representatives of the Institution of Civil Engineers, the Institution of Mechanical Engineers, the Institution of Naval Architects and the Institution of Electrical Engineers, was formed, with Mr. W. B. Worthington as its first Chairman.

The Joint Council, which is an advisory body without executive powers, has in the short time since its inception been engaged largely on matters affecting its own procedure. The Chairman and Secretary of the Joint Council will be provided by each of the four Founder Societies in rotation.

Roll of Honour.

On the 27th October, 1922, His Royal Highness the Duke of Connaught and Strathearn, K.G., unveiled the Roll of Honour which has been erected at the Institution in memory of the 346 members and Students who gave their lives in the Great War. The dedication service was conducted by the Dean of Westminster. A book of memoirs containing a record of these members and Students is in course of preparation, and it is expected that this volume will be available before the commencement of next session for those who wish to obtain it.

Wolfe Barry Memorial.

It may, at the same time, be recorded that a stained-glass window, which has been subscribed for by a number of friends and admirers of the late Sir John Wolfe Barry, and which has been placed in Westminster Abbey to perpetuate his memory, was, on behalf of the subscribers to the Memorial Fund, presented to the Abbey authorities by the President on the 7th December, 1922.

Geological Survey.

The Council desire to direct special attention to the valuable opportunities which may be given to the Geological Survey to confirm and investigate the geology of a particular district, if prompt notice is given to the Director of the Survey of any tunnelling operations, excavations, etc., which engineers may be carrying out.

THE ROLL.

Enumeration.

The Roll of the Institution on the 31st March, 1923, stood at 9,344, the changes which took place in it during the year ended on that date being shown in the following Table.

The Roll is, at this date, 9,365.

The Council record with special regret the deaths of Mr. Alexander Ross, Past-President, and Sir James Dewar, M.A., LL.D., F.R.S., Honorary Member.

The elections comprised 1 Honorary Member, 16 Members, 192 Associate Members, and 3 Associates; 377 candidates were admitted as Students, and the names of 2 Members, 14 Associate Members, 1 Associate, and 2 Students were restored to the Register. From this addition of 608 must be deducted the deaths, resignations and erasures during the year, the Associate elected an Honorary Member, the Students elected Associate Members, and those who, having passed the age-limit of the Student class, ceased to be con-

	1 April, 1921, to 31 March, 1922.						1 April, 1922, to 31 March, 1923.					
——	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.
Numbers at commencement . .	19	2312	5805	144	718	8998	19	2309	5861	125	879	9193
Transferred to Members	..	60	60	..	..		..	71	71	..	..	
Elections .	..	37	234	8	..	597	1	16	192	3	..	608
Admissions .	..	..	..	..	307		..	..	..	..	377	
Restored to Register .	..	3	8	..	..		..	2	14	1	2	
Deceased .	..	68	50	5	1	402	1	57	54	7	2	457
Resigned .	..	34	68	22	9		..	18	54	3	24	
Erased . .	..	1	5	..	1		..	9	59	3	11	
Elected to Honorary Member .	..	..	..	..	..		..	..	..	1	..	
Elected to Associate Members	..	..	..	..	25		..	..	..	..	20	
Removed—over age .	..	..	..	..	108		..	..	..	..	126	
Failed to complete election or admission	..	..	3	..	2		..	..	6	..	2	
						+195						+151
Numbers at termination	19	2309	5861	125	879	9193	19	2314	5823	115	1073	9344

nected with the Institution, together with those who failed to complete election or admission, amounting to 457 in all, showing a net addition of 151.

In accordance with the statement contained in last year's Annual Report, the Council would draw attention to the fact that they can no longer continue the privileges of Studentship to over-age Students who served with H.M. Forces during the war.

The full list of deaths is as follows (*E. refers to election, and T. to transfer*) :— Deaths.

Honorary Member (1).—*Sir* James Dewar, M.A., LL.D., F.R.S. (*E.* 1901).

Members (57).—William Murray Alston (*E.* 1902) ; Frederick Robert Bader (*E.* 1892 ; *T.* 1910) ; Jonathan Robert Baillie (*E.* 1888) ; James Bain (*E.* 1905) ; Francis Aloysius Bald (*E.* 1892 ; *T.* 1914) ; William Bates, O.B.E. (*E.* 1899 ; *T.* 1905) ; John Anderson Bensel (*E.* 1905) ; George Bohn (*E.* 1866 ; *T.* 1877) ; Alexander Joseph Bolton (*E.* 1887) ; Lalit Mohan Bose (*E.* 1904) ; Arthur Edward Brown (*E.* 1886 ; *T.* 1893) ; Alfred Edward Carey (*E.* 1877 ; *T.* 1884) ; William Alexander Cheeke (*E.* 1899 ; *T.* 1913) ; Alexander Cleghorn (*E.* 1892 ; *T.* 1903) ; Henry Aylwin Bevan

Cole (*E.* 1904); Daniel Connery, M.E. (*E.* 1891; *T.* 1898); Charles Augustus Copland (*E.* 1902; *T.* 1913); William Wallace Copland, M.V.O. (*E.* 1890; *T.* 1913); Leonard Creasy, I.S.O. (*E.* 1881; *T.* 1898); Benjamin Franklin Cresson, jun., (*E.* 1914); John D'Aeth, I.S.O. (*E.* 1888; *T.* 1896); Nelson Frederick Dennis (*E.* 1893; *T.* 1909); Philip Rickman Emmott (*E.* 1876; *T.* 1883); *Sir* John Gavey, C.B. (*E.* 1899); William Hay Gavin (*E.* 1915); William Bernard Godfrey (*E.* 1879; *T.* 1895); William Hall (*E.* 1858; *T.* 1881); Ashton Marler Heath (*E.* 1898; *T.* 1914); James William Helps (*E.* 1889; *T.* 1900); Arthur Edmund Breton Hill, B.A.Sc. (*E.* 1887; *T.* 1891); *Sir* Walter Charleton Hughes, C.I.E. (*E.* 1885); Henry Irwin, C.I.E. (*E.* 1875; *T.* 1879); James Thomas Jervis (*E.* 1893); James Johnston (*E.* 1888; *T.* 1898); Arthur William Lawder (*E.* 1878; *T.* 1880); Norman Joseph Lockyer (*E.* 1888; *T.* 1898); George Archibald Lundie (*E.* 1876; *T.* 1879); William Lunn (*E.* 1893; *T.* 1902); John Bower Mackenzie (*E.* 1875; *T.* 1878); Herbert Woodville Miller (*E.* 1892; *T.* 1899); William Julius Mirrlees, B.Sc. (*E.* 1891; *T.* 1895); Charles Ernest Norman (*E.* 1886; *T.* 1888); Walter Parry (*E.* 1887; *T.* 1895); *Hon.* Richard Clere Parsons, M.A. (*E.* 1876; *T.* 1883); James Noah Paxman (*E.* 1875; *T.* 1896); Lewis Henry Ransome (*E.* 1888; *T.* 1897); William Rich (*E.* 1904); Mark Heaton Robinson (*E.* 1893); Alexander Ross (*Past-President*) (*E.* 1880); Adam Scott (*E.* 1890; *T.* 1897); Joseph John Walklate (*E.* 1919); Herbert Wallis (*E.* 1880); Arthur Thomas Walmisley (*E.* 1874; *T.* 1886); Richard Saxton White (*E.* 1905); John Wilson (*E.* 1878); Henry Withy (*E.* 1904); Sydney William Yockney (*E.* 1876).

Associate Members (54).—Douglas John Alexander, B.Sc. (*E.* 1914); Shankar Govind Bam, B.E. (*E.* 1921); George Horatio Townshend Beamish (*E.* 1874); Alfred William Thomas Bean (*E.* 1880); Wilfrid Beloe (*E.* 1907); Nasarvanji Dorabji Bhada (*E.* 1886); Herbert Edwin Buckley (*E.* 1900); Cyrus Bullock (*E.* 1885); Henry Matthias Clarke (*E.* 1901); Alfred Harper Curtis, B.A. (*E.* 1890); Himatlal Dhirajram (Rao Bahadur) (*E.* 1899); Charles Arthur Richards Farrell, B.Sc. (*E.* 1908); Alexander MacArthur Finlayson (*E.* 1913); Leonard Macnamara Foulkes (*E.* 1906); Wiliam Henry Fry (*E.* 1886); William Gallon (*E.* 1884); John Jervis Garrard (*E.* 1891); Allen Gidley (*E.* 1914); Arthur Alexander Hall, B.A., B.E. (*E.* 1907); Richard James Hartley (*E.* 1886); John George Heslop (*E.* 1896); Arthur James Hillman, M.C. (*E.* 1914); William Francis Hole, B.E. (*E.* 1906); Alfred Hopkinson (*E.* 1877); Charles Edwin Jones (*E.* 1886); Walter William Murray Kitto (*E.* 1897); William Samuel Lackland (*E.* 1894); Geoffrey Lees (*E.* 1907); Henry Denzil Lobley, M.Sc. (*E.* 1915); Duncan William McArthur (*E.* 1882); Thomas McMaking (*E.* 1889); James Porter Maginnis (*E.* 1891); John Birch Minchin (*E.* 1881); Alexander James Neely, M.A., M.A.I. (*E.* 1898); James Napier Nicholson (*E.* 1904); John Shield Pattinson (*E.* 1885); Henry Charles Platts (*E.* 1892); Alfred Bertrand Potts (*E.* 1913); Dorabjee Bhikhajee Rabadina (*E.* 1891); John Charles Radford (*E.* 1889); Vernon Allen Ryder (*E.* 1919); Fred Simpson (*E.* 1879); T. Ridsdill Smith (*E.* 1895); John Stevenson (*E.* 1874); John Charles Taite (*E.* 1887); Herbert Edward Tasman (*E.* 1912); Robert Henry Taylor (*E.* 1886); Philip Walmesley Tolhurst (*E.* 1901); Nicholas Trestrail (*E.* 1894); William Donald Wilson White (*E.* 1896); Andrew Williamson, O.B.E.

(*E.* 1892) ; Frank Worrall (*E.* 1896) ; Wilfrid Wright (*E.* 1908) ; Albert Augustus Wynne (*E.* 1861.)

Associates (7).—George Corderoy (*E.* 1908) ; Léon Epstein (*E.* 1857) ; George Crispin Hammond, *Commander*, R.N. (*E.* 1878) ; William Bailey Hawkins (*E.* 1858) ; William Leighton Jordan (*E.* 1887) ; *Sir* William Lorimer (*E.* 1883) ; John Thomas Middleton (*E.* 1877).

Students (2).—James Reekie (*A.* 1921) ; Frederick Selwyn Walduck (*A.* 1921).

The following resignations have been received :—

Resignations.

Members (18).—James Barron ; Frederick Reily Collins ; Charles James Crowley ; Harry Dent ; James William Dugdale-Bradley ; Thomas Griffiths ; Edmund Mills Hann ; William Hawdon ; Henry Herbert Hellins ; Edward Hooper ; Herbert Vaughan Kent, C.B. ; Morice Leslie ; Charles Hesterman Merz ; Robert Moore ; William Robert Morris ; William Matthews Thomas ; Henry Seddon Wildeblood (*since reinstated*) ; John Peake Wildeblood.

Associate Members (54).—Lewis Brow Bagot ; William Banks ; Walter Francis Bayley ; Richard Michel Belhomme, B.A.I. ; Orient Bell ; Frederick Howie Blaine, B.A. ; Ernest John Vivian Bowman ; Edmund Henry Brietzcke ; Robert Crossley Bullough ; Edward Godfrey Carty, B.Sc. ; Henry Paul Ramsay Copeland ; Frank Whinfield Crawter ; James Crosbie ; Ralph Smith Oliver Cusack ; Augustine Campbell Davison ; Henry Francis Doran, B.A., B.A.I. ; William Macdonnell Mitchell Dowdell ; Arthur Sydney Welsh Elder ; Kentatsu Fujikura ; Fred Albert Gerrard ; Go Khek Ghee, B.Sc. ; Allan Greenwell ; John Gregson ; Thomas Hart ; Reginald Arthur Helps ; Thomas Blackburn Hennell ; Frank Geere Howard ; Stuart William Howard ; Frank Percy Jennings ; Adrian Hope Johnston ; Alexander Kerr ; James Thomas Landless ; James Peter McMillan ; John Harrison Middleton, B.A.I. ; Ralph Oakden, jun. ; Michael O'Brien, B.E. ; Charles Edmond Cleaver Peach, B.A. ; Frank Herbert Pinel ; George Victor Pont ; Frank Roberts ; John Harold Sagar ; William Campbell Shaw ; Charles Frederick Smith, D.Sc. ; William Tom Wood Somers ; James Henry Stephen ; Thomas Stewart ; Peter Anthony Thompson, B.A. ; Frank de Mierre Turner ; Thomas Henry Turney ; Alfred John Wadley ; Percy Warbrick, M.C. ; George Webb Ware ; Arthur Edward Williams, B.A. ; George William Williamson.

Associates (3).—Philip John Joseph Radcliffe, *Lt.-Col.*, R.E. ; *Sir* Alexander Rose Stenning ; Robert Stanser Templeton.

Students (24).—Charles Gaskell Alderton ; James Fleming Andrew ; Samuel Aston, B.Eng. ; Stuart Cecil Baillie ; Charles Biggar Blaikie ; Charles Wallace Chapman ; John Adair Coubrough, B.A. ; Leonard Schroeder Swinnerton Dyer ; Charles William Ellis ; Harold Victor Field ; Hugh Cowan Fraser, M.C. ; Edwin St. John Green ; Harry Hall ; Colton Richard Hinds ; Percy Patterson Jones ; Wallace Gordon Lindsay ; Henry Edward Merritt, B.Sc. ; Robert Davison Muir ; George Seymour Plant ; Cecil Norman Pye, B.Eng. ; George Montague Terence Rouse ; Norman Lancelot Stewart ; Charles Frederick Tissington, M.C. ; Kasper de Nyssen Wernecke.

FINANCE.

The following is a statement of receipts and payments for the year ended the 31st March, 1923, the account of which has been duly audited.

The Receipts were :—On General Income Account £40,783 18*s*. 0*d*., of which £36,628 12*s*. 11*d*. represents Subscriptions and Fees apportioned to the Financial Year, £1,044 8*s*. 5*d*., Dividends, Interest on Investments, etc., and £650 rent of No. 1, Great George Street.

The Expenditure was :—On General Account £40,370 13*s*. 1*d*., which includes Interest on Loan under amortization £982 14*s*. 8*d*., and Publications £8,717 7*s*. 0*d*.

There was received on Capital Account £2,414 15*s*. 0*d*., in Admission Fees, Life Compositions, and a Legacy under the will of the late Mr. Charles Ower, Assoc. M. Inst. C.E., and £971 12*s*. 1*d*. was applied to the extinguishment of the Loan.

On Trust Funds Income Account there was received a total of £1,357 9*s*. 0*d*., and the Expenditure amounted to £1,616 11*s*. 6*d*.

Of the balance £1,450 19*s*. 8*d*. brought forward from last year, the sum of £1,106 14*s*. 11*d*. was expended on the investigation into the Deterioration of Structures exposed to Sea Action.

THE PUBLICATIONS.

Important changes in regard to the Institution publications—partly dictated by financial considerations, but also adopted with a view to meet other changed conditions, and to keep all members fully and regularly informed as to the Institution's activities—have been made during the Session.

Briefly, these changes consist in the limitation of the "Minutes of Proceedings" to the records of Institution Meetings; the publication of Papers which are not discussed in the form of separate pamphlets, circulated to those members who apply for them; and the issue of an additional publication ("Sessional Notices") circulated to the whole of the membership several times during the sessional period.

The Council believe that the new arrangements will commend themselves to the members, and that the publications as a whole will be found better adapted to secure the earliest practicable publication of Original Communications and other information.

The "Sessional Notices," while obviously not intended to have the permanent character of the Proceedings, will be so arranged and indexed as to facilitate the maintenance of a file of several years, even if members do not elect to bind them. With regard to the Abstracts of forthcoming Papers published in them, in announcing the arrangements a hope was expressed that members abroad would often find themselves able to contribute written observations on the subjects of the Papers on the basis of these Abstracts. It is realized, of course, that in the case of a controversial Paper members will usually desire to see the full text of the Paper, and will apply for an advance proof; but besides forming a medium for criticism and expression of personal views, the "Correspondence" has always consisted largely of valuable records of executed work and of experience; and for such matters an Abstract may equally afford occasion for the communication of information supplementing the Paper. It is to be borne in mind that the governing factor as regards prompt publication of the Minutes of Proceedings is the time necessary for the reception and preparation of contributions to the discussions from members abroad; so that any step which will reduce that time is of advantage to the whole of the members.

THE SESSIONAL PROCEEDINGS.

Papers and Discussions.

Sixteen Ordinary Meetings have been held during the Session, five of which were for the purpose of continuing the discussion of Papers. The Inaugural Address of the President, Dr. W. H. Maw, was delivered at the first of these Meetings, and at the others seventeen Papers have been read and discussed, of which the following is a list:—

AUTHOR.	TITLE.
E. O. Forster Brown.	Underground Waters in the Kent Coal-field and their Incidence in Mining Development.
W. A. Fraser, M. Inst. C.E.	Strengthening of the Floor, etc., of the Forth Bridge.
F. M. G. Du-Plat-Taylor, M. Inst. C.E.	Extensions at Tilbury Docks, 1912–1917.
H. W. H. Richards.	Twelve Years' Operation of Electric Traction on the London, Brighton and South Coast Railway.
Harold Stringer, B.A., Assoc. M. Inst. C.E.	Railway Economics for Extra-European Railways. Reinforced Concrete on the Chinese Railways.

D. H. Remfry, Assoc. M. Inst. C.E.	Wind-Pressures, and Stresses caused by the Wind on Bridges.
F. W. Jameson.	Sewage-Disposal in South Africa, with special Reference to Sludge-Treatment at Pretoria.
R. E. Tickell, O.B.E., M. Inst. C.E.	Colombo Drainage-Works.
D. E. Lloyd-Davies, M. Inst. C.E.	Main Drainage of the Southern Suburbs of the City of Capetown, South Africa.
	The Works for the Augmentation of the Supply of Water to the City of Capetown, South Africa.
J. M. Lacey, M. Inst. C.E.	Some Problems connected with the Rivers and the Canals in Southern India.
S. L. Rothery, Assoc. M. Inst. C.E.	An Irrigation Project of the Californias.
A. B. Buckley, jun., O.B.E., Assoc. M. Inst. C.E.	The Influence of Silt on the Velocity of Water Flowing in Open Channels.
J. C. Ross, M. Inst. C.E.	The Improvement of the Water-Supply to the City of Hobart, Tasmania; and the Manufacture of Reinforced-Concrete Pipes by the Centrifugal Process.
Mark Randall, Assoc. M. Inst. C.E.	The Mazoe Irrigation-Dam.
Asa Binns, M. Inst. C.E.	The King George V Dock, London.

Awards.

For some of the Communications considered at Ordinary Meetings the Council have made awards to Messrs. Richards, Forster-Brown, Binns, Buckley, Fraser, Remfry, Randall, Lloyd-Davies, Du-Plat-Taylor and Jameson.

Informal Meetings.

In accordance with the arrangements announced in October last, four Informal Meetings have been held during the Session, with the object of providing opportunities for the younger members to debate upon engineering matters. The subjects introduced and the names of the introducers are as follows :—

Railway Maintenance : Examples of Wear and Defects occurring in Structures and Track. A. Wood-Hill, M. Inst. C.E.
New Factors in the Reconditioning of Trunk Highways and their Bridges. R. G. H. Clements, Assoc. M. Inst. C.E.
A Comparison of American and Canadian Methods of Engineering with English. G. W. M. Boycott, Assoc. M. Inst. C.E.
Electric Train Lighting. J. R. W. Grainge.

Whilst the attendances have not been—and perhaps cannot be expected to be—large, it is believed that the discussions have been of interest to those who have attended.

LECTURES.

The Vernon-Harcourt Lectures were delivered in London on the 17th and 31st January by Mr. T. R. Wilton, M.A., M. Inst. C.E., on the subject of "Foundations in Dock and Harbour Works." These Lectures have been delivered also before the Local Associations at Manchester, Newcastle, and Glasgow.

The Council arranged also for two Institution Lectures to Students on "Engineering Factory Organization," to be given by Sir Henry Fowler, K.B.E., M. Inst. C.E. These Lectures were delivered in London on the 29th November and 6th December, and were repeated at Leeds, Birmingham, and Bristol.

A Students' Lecture on "Recent Developments in Wireless Telephony" was delivered by Captain H. Riall Sankey, C.B., C.B.E., M. Inst. C.E., in London on the 28th February.

The Twenty-Ninth James Forrest Lecture is to be delivered by Sir Richard T. Glazebrook, K.C.B., F.R.S., Hon. M. Inst. C.E., on Friday the 4th May.

STUDENTS' MEETINGS AND VISITS.

In addition to the five Meetings of London Students at which the above-mentioned lectures were delivered, four Ordinary Meetings have been held ; at the first of these the Chairman of the London Students' Committee delivered an Address, and at the remainder

Papers were read and discussed. One Informal Meeting was held, and seven Visits have been paid to places of engineering interest. The London Students' Forty-Second Annual Dinner was held on the 23rd March.

THE LIBRARY.

During the year 277 volumes were presented and 42 were purchased, making a total, on the 31st March, 1923, of 51,215.

The arrangements already announced in the "Sessional Notices" for the loan, to members and Students in Great Britain and Ireland, of such current engineering text-books as can readily be procured, are working satisfactorily. The scheme enables the Institution to place at the service of members the latest editions of such books, and to build up gradually a loan collection, without involving a heavy initial outlay on books, some of which might never be asked for.

For extending the very large and valuable collection of engineering pamphlets and reports in the reference Library, the Institution is very largely dependent upon the good offices of members; and the Council will be particularly indebted to any members who, when disposing of such literature, will afford the Institution an opportunity of adding to its collection.

It is gratifying to record that the use of the reference Library by members is steadily growing. The financial stringency has necessarily curtailed expenditure on the acquisition of new books; but on the other hand, many new members, finding themselves, under the prevailing conditions, without suitable material for the preparation of an original communication, have discharged the obligation entered into on election by presenting a book or books to the Library; and the number of books so acquired in the last 2 or 3 years has been so large as to enable the Library to keep abreast of the publication of new engineering works and editions in the English language.

The following additions have been made during the year to the paintings, engravings and drawings in the possession of the Institution:—A portrait, by Miss S. H. Purser, R.H.A., of Sir John P. Griffith, Past-Pres. Inst. C.E., presented by himself; a portrait, by George Harcourt, A.R.A., of Mr. W. B. Worthington, Past-Pres. Inst. C.E., presented by himself; a photogravure of the Hon. Sir Charles A. Parsons, M. Inst. C.E., from the portrait by Sir William Orpen, R.A., presented by the Parsons Marine Steam Turbine Company; water-colour drawings of Parliament Square, Westminster, and Richmond, Yorkshire, by William Berrell, M. Inst. C.E., bequeathed by Miss J. H. Berrell; the Royal Border Bridge, by Sir Thomas Pittar, K.C.B., presented by Mr. G. F. Pittar, Assoc. M. Inst. C.E.; and a drawing of an Experimental Filter Bed, 1829, signed by James Simpson, Past-Pres. Inst. C.E., presented by Mr. C. Liddell Simpson, Assoc. M. Inst. C.E. There has also been received, from Mr. H. M. Rootham, Assoc. M. Inst. C.E., a medal struck to commemorate the opening of the Thames Tunnel in 1845.

APPEN

BALANCE SHEET

	£	s.	d.	£	s.	d.
To INSTITUTION CAPITAL ACCOUNT AND BUILDING FUND, *as detailed on pages 444 & 445*	..			348,591	3	4
„ LOAN on security of Institution Buildings	..			22,553	16	6
„ CREDITORS AND CREDIT BALANCES	..			1,127	9	3
„ ROLL OF HONOUR FUND	..			34	1	11
„ TRUST FUNDS, CAPITAL AND INCOME ACCOUNTS—						
Capital Accounts—*as detailed on page 448, invested per contra*	36,953	16	0			
Income Accounts—Balances unexpended—*as detailed on page 451*	3,364	1	3			
				40,317	17	3
„ INSTITUTION REVENUE IN SUSPENSE—						
Proportion of 1923 Subscriptions applicable to the nine months from 1st April to 31st December, 1923	15,928	12	1			
Subscriptions received in advance	69	19	7			
				15,998	11	8
				£428,622	19	11

AUDITORS

We have audited the above Balance Sheet dated 31st March, 1923, and have obtained
is properly drawn up so as to exhibit a true and correct view of the state of The Institution's
shown by the books of The Institution.

London, 23 April, 1923.

DIX I.

31st MARCH, 1923.

				£ s. d.
By **Expenditure on Institution Buildings, including Cost of Site**, *as per last account*				352,072 7 2
„ **Institution Investments** at cost, *as detailed on page 448*				26,623 15 7
Note.—*The value of these Investments at ruling prices on 31st March, 1923, amounted approximately to £21,575.*				
„ **Trust Funds Investments, etc.**—				
Capital:—				
Investments, *as detailed on page 449*		£36,921 12 0		
Cash at Bank		32 4 0	36,953 16 0	
Unexpended Income:—				
Investments, *as detailed on page 449*		£529 16 1		
Cash at Bank—				
On Deposit a/c	2,700 0 0			
„ Current a/c	134 5 2	2,834 5 2	3,364 1 3	40,317 17 3
„ **Sundry Debtors**				443 16 9
„ **Cash at Bankers, Deposit and Current Accounts**—				
Institution Funds			8,761 2 2	
Roll of Honour Fund			34 1 11	
Sea Action Committee Account			369 19 1	9,165 3 2
Note.—*No value has been attached, for the purpose of this Balance Sheet, to the Books, Furniture, Pictures, Sculpture, etc., belonging to The Institution.*				
				£428,622 19 11

H. H. Jeffcott, *Secretary.*

PORT.

the information and explanations we have required. In our opinion such Balance Sheet
irs according to the best of our information and the explanations given to us, and as

Percival D. Griffiths, F.C.A.
Maurice F.-G. Wilson.
} Auditors.

INSTITUTION CAPITAL ACCOUNT AND BUILDING

	£	s.	d.
To Balance *carried down*	348,591	3	4
	£348,591	3	4

FUND, 31st MARCH, 1923.

		£	s.	d.
By INSTITUTION CAPITAL ACCOUNT AND BUILDING FUND, *as per last account*		345,763	3	5
,, RECEIPTS DURING THE YEAR TO 31ST MARCH, 1923—				
Admission Fees	£2,278 10 0			
Life Compositions	126 5 0			
Legacy under the Will of the late Charles Ower, Assoc. M. Inst. C.E.	10 0 0	2,414	15	0
,, INSTITUTION REVENUE ACCOUNT—				
Excess of Income over Expenditure for the year to 31st March, 1923, *page 446*		413	4	11
		£348,591	3	4
By BALANCE *brought down—as per Balance Sheet, page 442*		£348,591	3	4

INSTITUTION REVENUE ACCOUNT

EXPENDITURE.

				£	s.	d.	£	s.	d.
To HOUSE AND ESTABLISHMENT CHARGES—									
Rates, Taxes and Insurance				7,290	8	9			
Electric Lighting and Power, Water Supply, Warming, Ventilating and Telephone				920	15	0			
Cleaning and Household Expenses				1,526	5	6			
Refreshments and Assistance at Meetings				146	15	8	9,884	4	11
,, REPAIRS AND RENEWALS TO STRUCTURE, FURNITURE, FITTINGS AND MACHINERY				..			553	18	1
,, SALARIES, WAGES AND RETIRING ALLOWANCES—									
Salaries (including Bonus)				4,115	0	0			
Retiring Allowances				1,605	0	0			
Clerks, Messengers and Housekeeper (including Bonus)				4,932	14	9	10,652	14	9
,, PREMIUMS ON POLICIES FOR STAFF PENSIONS—Proportion paid by the Institution							882	17	8
,, STATIONERY, POSTAGES, ETC.—									
Stationery and Printing				2,149	7	3			
Postages, Telegrams and Parcels				857	15	10	3,007	3	1
,, PUBLICATIONS—	£	s.	d.						
Minutes of Proceedings	6,121	17	10						
Sessional Notices	189	3	8						
Subject-Index: Vols. 205–212	23	4	0						
Charters, By-laws and Lists of Members	370	18	6	6,705	4	0			
Abstracts	1,865	7	0						
Less Repayment of Transmission Charges	359	4	0						
	1,506	3	0						
Clerical Pay	506	0	0	2,012	3	0	8,717	7	0
,, LIBRARY—									
Books and Periodicals				309	16	6			
Binding				39	1	0			
Clerical Charge (previously included under "Salaries")				600	0	0	948	17	6
,, EXAMINATION EXPENSES—									
Examiners, Printing and General				1,215	11	5			
Clerical Pay				766	8	0	1,981	19	5
,, CONVERSAZIONE AND ANNUAL DINNER				..			1,188	12	9
,, DIPLOMAS AND MEDALS—									
Diplomas				60	18	6			
Stephenson and Watt Medals				33	5	6	94	4	0
,, LOCAL ASSOCIATIONS—									
Grants to Local Associations, etc.				..			741	17	6
,, CONTRIBUTIONS TOWARDS ADVISORY COMMITTEES IN THE COLONIES				..			205	0	0
,, HEAT ENGINE AND BOILER TRIALS COMMITTEE				..			2	19	7
,, GRANTS AND CONTRIBUTIONS—									
Conjoint Board of Scientific Societies				5	5	0			
Westminster Hospital				10	10	0	15	15	0
,, LEGAL AND OTHER PROFESSIONAL CHARGES—									
Legal Charges				327	11	2			
Auditor's Fee				157	10	0	485	1	2
,, DEDICATION SERVICE WESTMINSTER ABBEY—SIR JOHN WOLFE BARRY MEMORIAL WINDOW				..			25	6	0
,, INTEREST ON LOANS				..			982	14	8
							40,370	13	1
,, BALANCE CARRIED TO CAPITAL ACCOUNT				..			413	4	11
							£40,783	18	0

FROM 1st APRIL, 1922, TO 31st MARCH, 1923.

INCOME.

	£	s.	d.	£	s.	d.
By SUBSCRIPTIONS APPLICABLE TO THE FINANCIAL YEAR 1922-1923	..			35,288	16	11
,, MEMBERSHIP FEES	..			1,339	16	0
,, INTEREST, DIVIDENDS, ETC.—						
On Institution Investments	879	0	0			
On Deposit	21	8	5			
Income Tax Refunded for the year 1921-22	144	0	0			
				1,044	8	5
,, EXAMINATION FEES	..			2,104	7	6
,, MINUTES OF PROCEEDINGS (REPAYMENT FOR BINDING)	..			283	14	2
,, LIBRARY FUND DONATIONS	..			72	15	0
,, RENT OF 1, GREAT GEORGE STREET *(Tenancy expires in 1925, when the building will be demolished.)*	..			650	0	0
				£40,783	18	0

TRUST FUND CAPITAL ACCOUNTS AT 31st MARCH, 1923.

	£	s.	d.
TELFORD FUND	7,988	9	4
MANBY DONATION	270	0	0
MILLER FUND	4,850	2	4
HOWARD BEQUEST	500	0	0
TREVITHICK MEMORIAL	100	0	0
CRAMPTON BEQUEST	500	0	0
JAMES FORREST LECTURE AND MEDAL FUND	1,104	14	0
PALMER SCHOLARSHIP FUND	1,430	18	0
JOHN BAYLISS BEQUEST	1,000	0	0
THE INDIAN FUND	1,148	11	8
VERNON-HARCOURT BEQUEST	1,000	0	0
WEBB BEQUEST	1,000	0	0
YARROW EDUCATIONAL FUND	10,049	9	6
WILLIAM LINDLEY FUND	1,435	4	9
KELVIN MEDAL FUND	475	0	0
CHARLES HAWKSLEY BEQUEST	3,000	0	0
COOPERS HILL WAR MEMORIAL FUND	1,101	6	5
As per Balance Sheet, page 442	£36,953	16	0

INSTITUTION INVESTMENTS AT 31st MARCH, 1923.

£	s.	d.		COST. £	s.	d.
3,000	0	0	Metropolitan Water Board "B" 3% Stock	2,958	16	0
6,000	0	0	London and North Eastern Railway 4% Debenture Stock (Conversion of £6,000 Great Eastern Railway 4% Debenture Stock)	7,749	18	3
6,000	0	0	London, Midland and Scottish Railway 4% Debenture Stock (Conversion of London and North Western Railway 3% Debenture Stock)	7,452	14	8
4,700	0	0	War Loan 5% 1929-1947	4,462	6	8
2,000	0	0	National War Bonds 1924, 1st Series	2,000	0	0
2,000	0	0	„ „ „ 4th Series	2,000	0	0
			As per Balance Sheet, page 443	£26,623	15	7

NOTE.—*The value of these Investments at ruling prices on 31st March, 1923 amounted approximately to £21,375.*

TRUST FUNDS INVESTMENTS AT 31ST MARCH, 1923.

	Capital.			*Unexpended Income.*		
	£	s.	d.	£	s.	d.
TELFORD FUND.—£8,738 13*s.* 0*d.* 2½% Consols	7,988	9	4			
MANBY DONATION. £250 London North Eastern Railway 4% 2nd Guaranteed Stock (conversion of £250 G. E. Rly. 4% Guar. Stock)	270	0	0			
MILLER FUND. £5,129 17*s.* 5*d.* 2½% Consols	4,850	2	4			
£250 War Stock 1929–1947 *Unexpended Income*				237	10	0
HOWARD BEQUEST.—£551 14*s.* 6*d.* 2½% Consols	500	0	0			
TREVITHICK MEMORIAL.—£103 2½% Consols	100	0	0			
CRAMPTON BEQUEST.—£512 15*s.* 11*d.* 2½% Consols	500	0	0			
JAMES FORREST LECTURE AND MEDAL FUND. £465 Southern Railway 4% Debenture Stock (conversion of £372 S. E. Rly. 5% Debenture Stock £500 5% War Stock 1929–1947	1,073	10	0			
PALMER SCHOLARSHIP.—£1,496 6*s.* 1*d.* Metropolitan 3% Stock	1,430	18	0			
JOHN BAYLISS BEQUEST. £1,013 17*s.* 10*d.* London County 3% Stock	1,000	0	0			
THE INDIAN FUND.—£1,353 4*s.* 2*d.* 2½% Consols	1,148	11	8			
VERNON-HARCOURT BEQUEST. £1,082 9*s.* 10*d.* London County 3% Stock	1,000	0	0			
WEBB BEQUEST. £1,055 7*s.* 2*d.* Metropolitan Water Board 3% "B" Stock	1,000	0	0			
YARROW EDUCATIONAL FUND. £4,205 London, Midland & Scottish Railway 4% Preference Stock (conversion of £6,728 Mid. Rly. 2½% Pref. Stock) £2,110 London & North Eastern Railway 4% 1st Preference Stock £2,110 London & North Eastern Railway 4% 2nd Guaranteed Stock (conversion of £4,220 N. E. Rly. 4% Pref. Stock)	10,049	9	6			
WILLIAM LINDLEY FUND. £1,000 10*s.* 0*d.* London, Midland & Scottish Railway 4% Debenture Stock (conversion of £1,334 L. & N. W. Rly. 3% Deb. Stock	1,435	4	9			
£213 15*s.* 0*d.* London, Midland & Scottish Railway 4% Deb. Stock (conversion of £285 L. & N. W. Rly. 3% Deb. Stock) *Unexpended Income*				149	16	1
£150 5% War Stock 1929–1947 *Unexpended Income*				142	10	0
KELVIN MEDAL FUND. £500 5% War Stock 1929–1947	475	0	0			
CHARLES HAWKSLEY BEQUEST. £955 Metropolitan Water Board 3% "B" Stock £500 South Essex Waterworks 5% Preference Stock £70 5% Sheffield Corporation Annuities £40 4% " " "	3,000	0	0			
COOPERS HILL WAR MEMORIAL. £1,238 15*s.* 1*d.* 5% War Stock 1929–1947	1,100	6	5			
As per Balance Sheet page 443	£36,921	12	0	529	16	1

NOTE.—*The value of these Investments at ruling prices on 31st March, 1923, amounted approximately to £26,927 and £594 respectively.*

TRUST FUNDS INCOME ACCOUNTS FROM

—	Balance at 1st April, 1922.		
	£	s.	d.
Telford Fund	146	18	5
Manby Fund	27	12	11
Miller Fund	999	2	0
Howard Bequest	76	1	2
Trevithick Memorial	16	5	8
Crampton Bequest	58	12	4
James Forrest Lecture and Medal Fund	120	4	10
Palmer Scholarship Fund	195	4	3
John Bayliss Bequest	78	6	10
Indian Fund	110	7	6
Vernon-Harcourt Bequest	177	19	10
Webb Bequest	129	4	9
Yarrow Educational Fund	464	18	5
William Lindley Fund	336	13	8
Kelvin Medal Fund	99	19	10
Charles Hawksley Bequest	554	12	0
Coopers Hill War Memorial Fund	30	19	4
Totals	3,623	3	9

1st APRIL, 1922, TO 31st MARCH, 1923.

Income (including Income Tax refunded for the year 1921–22).			Expenditure: Scholarships, Prizes, Lectures, etc.			Balance at 31st March, 1923.		
£	s.	d.	£	s.	d.	£	s.	d.
219	1	7	319	18	9	46	1	3
10	10	11	25	0	0	13	3	10
154	16	6	120	4	6	1,033	14	0
14	19	9	..			91	0	11
2	12	6	15	12	0	3	6	2
12	18	8	59	1	0	12	10	0
45	17	5	47	12	0	118	10	3
46	15	4	100	0	0	141	19	7
31	9	5	30	0	0	79	16	3
35	7	2	30	0	0	115	14	8
34	5	9	75	0	0	137	5	7
33	16	6	..			163	1	3
384	10	10	711	13	3	137	16	0
62	14	0	37	10	0	361	17	8
26	13	6	..			126	13	4
177	14	10	45	0	0	687	6	10
63	4	4	..			94	3	8
1,357	9	0	1,616	11	6	3,364	1	3
						As per Balance Sheet, p. 442.		

ACCOUNT OF THE COMMITTEE ON THE DETERIORA-

FROM 1ST APRIL, 1922,

	£	s.	d.
To Amount expended during the year to 31st March, 1923. . .	1,106	14	11
„ Balance at 31st March, 1923	369	19	1
	£1,476	14	0

ROLL OF HONOUR

FROM 1ST APRIL, 1922,

	£	s.	d.
To Amount expended during the year to 31st March, 1923 . .	3,595	13	0
„ Balance at 31st March, 1923	34	1	11
	£3,629	14	11

TION OF STRUCTURES EXPOSED TO SEA ACTION,

TO 31ST MARCH, 1923.

	£	s.	d.
By Balance at 31st March, 1922, per last Account	1,450	19	8
,, Interest on Deposit	25	14	4
	£1,476	14	0
,, Balance at 31st March, 1923, *brought down*	£369	19	1

FUND ACCOUNT,

TO 31ST MARCH, 1923.

	£	s.	d.
By Balance at 31st March, 1922	3,117	7	4
,, Subscriptions	462	16	0
,, Interest on Deposit	49	11	7
	£3,629	14	11
,, Balance at 31st March, 1923, *brought down*	£34	1	11

APPENDIX II.

MEMORANDUM OF THE FINANCIAL POSITION OF THE INSTITUTION, WITH SPECIAL REFERENCE TO THE QUESTION OF THE REVISION OF SUBSCRIPTION RATES.

With reference to the Resolution passed at the Special General Meeting on the 26th April, 1921, as affecting the subscriptions and fees subsequent to 1923, the Council now place before The Institution particulars of the financial position at the present time.

In October, 1920, and January, 1921, the Council informed The Institution, that, having given careful consideration to every means whereby economies might be effected, they had to face an approximate increase on the normal expenditure of some £10,000 to £11,000 per annum, due to the heavy advance in rates and taxes, charges for heating, lighting, ventilating, printing and postage, etc.

The Council estimated that the revised scale of subscriptions would produce approximately an additional £12,000 annually, less an uncertain amount due to the subscriptions of those members who had retired from business remaining at the old rate. This deduction eventually proved to be some £1,500, in respect of 500 to 600 members.

The accompanying comparative statement will show that the economies which have been effected during the year 1922–1923, and the fall in printing and other costs, have resulted in a substantial reduction in expenditure as compared with the preceding year; nevertheless, at the present time the expenditure is still some £13,000 in excess of that of 1914–1915, the last year of approximately normal working.

It may be observed that, at the time the question of the increase of subscriptions was engaging The Institution's attention, the Building was in process of being re-assessed for district rating purposes. The sum which it was estimated The Institution would have to pay under the heading of "Rates, Taxes and Insurance" was £5,279 as against £3,569 in 1914–1915, but, owing to the rateable value of the Institution building being increased from £8,750 to £11,459, the actual charges under this heading have been, during the past two years, £7,744 15*s.* 1*d.* and £7,290 8*s.* 9*d.*, respectively, the last-named sum being £3,721 1*s.* 3*d.* higher than the corresponding charges for the year 1914–1915.

Expenditure under the heading of "House and Establishment Charges" has been reduced by some £858 during the past year.

"Repairs and Renewals, etc.," are down to £553 18*s.* 1*d.*, against £1,491 11*s.* 7*d.* in 1914–1915.

With regard to "Pension Fund and Retiring Allowances," the actual retiring allowances are little more than in 1914–1915, the increase under this heading being practically due to the fact that the Council have found it desirable to make provision for the future, instead of leaving these matters to be met out of current revenue.

During the Financial Year just concluded, alterations in the method of issuing notices of meetings and other matters, the fall in printing costs generally, and lower postal rates, have resulted in a reduction of some £1,200 under the heading of "Stationery and Postages."

Since April, 1921, the remaining delayed Volumes of Proceedings (publication of which was held over owing to war conditions) have been printed and issued. The cost of this work has been met partly by a charge against subsequent income and partly by the accumulated excess of Income over Expenditure for the years 1917–1918 to 1919–1920, due to the curtailment of The Institution's operations in several directions during the war period, and further to its establishment charges being shared by Government Departments accommodated in its premises at that time.

The Examination expenses have increased, but this is met by the augmented receipts from Examination Fees.

The expenditure on the Local Associations, the activities of which were partially suspended during the War, has lately increased and has reached its pre-war level.

The net increase in the subscription revenue during the past year (1922–1923) was approximately £11,000, making an increase of 45 per cent. on the old subscription revenue, whilst expenditure was some 46 per cent. over 1914–1915 figures.

It will be noted that no provision has been made in the way of a Depreciation Fund for possible future large renovations of the structure of the Building or the Plant.

As will be seen from the Annual Report, the Council have, during the past year, taken certain steps which it is hoped will develop and increase the service of The Institution to the Civil Engineering Profession and which will not, it is thought, increase materially its expenditure.

It will be seen from the foregoing that, under the present conditions, an undiminished income is essential to meet the current expenditure, and that a reduction in the subscription rates could not be entertained at present without seriously curtailing the maintenance and extension of the activities of The Institution.

The Council do not anticipate that during the next year or so there will be any important amelioration of the financial position as it stands at present. Should, however, any small balance be available it would be used towards restoring the income account to its normal condition, and thus act as a partial set-off against the adverse balances of 1920–1921 and 1921–1922.

The Council will continue to use every endeavour to effect all possible financial economies, and the members may rely on their proposing a reduction of the subscription charge so soon as they find it may wisely be accomplished.

19th April, 1923

THE INSTITUTION OF

INCOME AND EXPENDITURE—FINANCIAL YEARS (1st APRIL-

Expenditure.	1914-1915			1920-1921			1921-1922			1922-1923		
	£	s.	d.	£	s.	d.	£	s.	d.	£	s.	d.
To Rates, Taxes and Insurance	3,569	7	6	5,622	14	11	7,744	15	1	7,290	8	9
,, House and Establishment Charges . . .	2,030	14	4	2,824	3	2	3,451	19	5	2,593	16	2
,, Repairs and Renewals to the Premises, Furniture, Fittings and Machinery	1,491	11	7	2,988	0	4	1,617	10	4	553	18	1
,, Staff Salaries	3,687	10	0	4,269	4	9	3,884	0	0	4,115	0	0
,, Clerks', Messengers' and Attendants' Wages . .	2,362	14	6	5,789	4	7	5,578	3	10	4,932	14	9
,, Pension Fund and Retiring Allowances . .	1,320	0	0	975	3	3	1,281	5	3	2,487	17	8
,, War Allowances to Clerks, etc., on Military Service	273	4	0	..			..			..		
,, Stationery, postages, etc..	2,245	13	4	6,200	8	6	4,238	13	5	3,007	3	1
,, Publications	5,822	1	11	15,392	10	4	18,104	6	8	8,717	7	0
,, Library	358	6	0	322	18	8	399	11	0	948	17	6
,, Examination Expenses .	946	5	5	1,487	5	11	1,825	17	8	1,981	19	5
,, Conversazione, Annual Dinner	1,001	9	4	1,241	11	10	..			1,188	12	9
,, Engineering Conference.	..			..			680	6	11	..		
,, Local Associations . .	179	7	11	940	1	3	478	13	8	741	17	6
,, Expenses of Special Committees and Advisory Committees . .	14	14	5	160	0	0	145	0	0	205	0	0
,, Grants, Contributions, etc.	510	10	0	10	10	0	60	10	0	15	15	0
,, Diplomas, Medals and Prizes	63	15	0	62	6	6	86	1	0	94	4	0
,, Legal and Auditors' Charges	193	17	10	1,142	6	8	862	1	10	485	1	2
,, Interest on Loan . .	1,453	2	4	1,044	4	10	1,012	15	1	982	14	8
,, Memorial Services . .	..			..			24	12	6	25	6	0
,, Expenses of Heat Engine Committee . .	..			..			..			2	19	7
,, War Memorial Preliminary Expenses . . .	..			150	0	0	..			..		
Total Expenditure . .	27,524	5	5	50,632	15	6	51,476	3	8	40,370	13	1
,, Balance (excess of Income)	728	17	10	..			..			413	4	11
	28,253	3	3	50,632	15	6	51,476	3	8	40,783	18	0

CIVIL ENGINEERS.

31ST MARCH) 1914-1915, 1920-1921, 1921-1922 AND 1922-1923.

Income.	1914-1915			1920-1921			1921-1922			1922-1923		
	£	s.	d.	£	s.	d.	£	s.	d.	£	s.	d.
By Subscriptions applicable to the Financial Year .	23,868	2	2	24,109	5	11	32,595	6	2	35,288	16	11
,, Membership Fees . .	1,029	0	0	1,459	10	0	1,478	8	0	1,339	16	0
,, Interest, Dividends, etc.	752	5	0	2,012	13	2	1,305	0	0	1,044	8	5
,, Examination Fees . .	1,110	10	6	1,555	1	0	2,062	4	0	2,104	7	6
,, Minutes of Proceedings (Repayment for Binding)	632	8	3	329	4	6	237	7	0	283	14	2
,, Library Fund Donations	172	19	0	90	11	0	88	11	6	72	15	0
,, Rent of No. 1, Great George Street (terminating in 1925) . . .	687	18	4	650	0	0	650	0	0	650	0	0
,, Contributions received from Government Departments towards expenses of Temporary Accommodation afforded in the Institution Building and its subsequent reinstatement	..			3,463	19	5	..			..		
Total Receipts . . .	28,253	3	3	33,670	5	0	38,416	16	8	40,783	18	0
,, Balance (Excess of Expenditure	..			16,962	10	6	13,059	7	0	..		
	28,253	3	3	50,632	15	6	51,476	3	8	40,783	18	0

MEDALS AND PREMIUMS AWARDED,

SESSION 1921–1922.

FOR PAPERS READ AND DISCUSSED AT THE ORDINARY MEETINGS.

1. Telford Gold Medals to Sir Henry Fowler,[1] K.B.E., M. Inst. C.E., and Herbert Nigel Gresley, C.B.E., M. Inst. C.E., for their joint Paper on "Trials in Connection with the Application of the Vacuum-Brake for Long Freight-Trains."
2. A Telford Gold Medal to Horace Field Parshall,[2] D.Sc., M. Inst. C.E., for his Paper on "Hydro-Electric Installations of the Barcelona Traction, Light and Power Company."
3. A Watt Gold Medal to William Willox,[3] M.A., M. Inst. C.E., for his Paper on "All-Electric Automatic Power Signalling on the Metropolitan Railway."
4. The Indian Premium to Frederick George Royal-Dawson, M. Inst. C.E., for his Paper on "The Indian Railway Gauge Problem."
5. A Telford Premium to Alan Wood Rendell, M. Inst. C.E., for his Paper on "Control of Trains, in relation to increased Weight and Speed combined with reduced Headway."
6. Telford Premiums to Arthur Cliffe Walsh,[4] Assoc. M. Inst. C.E., and William Frank Stanton, B.Sc., Assoc. M. Inst. C.E., for their joint Paper on "The Improvement of the Port of Valparaiso."

FOR PAPERS PRINTED IN SECTION II OF THE PROCEEDINGS.

1. A George Stephenson Gold Medal to Bernard Courtney Laws, D.Sc., M. Inst. C.E., for his Paper on "Distribution of Stress in Thin Mild-steel Plates of Rectangular Shape, fixed along their Edges, and subject to Uniformly-distributed Loads."

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[1] Has previously received a Watt Medal, a Telford Premium, a Webb Prize, and a Miller Prize.

[2] Has previously received a Crampton Prize and a Telford Premium.

[3] Has previously received a Telford Premium

[4] Has previously received a Miller Prize.

2. A Telford Premium jointly to Professor Leonard Bairstow, C.B.E., F.R.S., and Alfred John Sutton Pippard,[1] M.B.E., D.Sc., Assoc. M. Inst. C.E., for their Paper on "The Determination of Torsional Stresses in a Shaft of any Cross Section."
3. A Telford Premium to Edward Alexander Cullen, M. Inst. C.E., for his Paper on "The Improvement of the Brisbane River."
4. A Telford Premium to Henry Harvey Dare,[2] M.E., M. Inst. C.E., for his Paper on "The Spillways of Burrinjuck Dam."
5. A Telford Premium to Frederick William Stephen, M.C., Assoc. M. Inst. C.E., for his Paper on "The Nile Water-Supply to Palestine for the Egyptian Expeditionary Force."

For Papers Read Before Meetings of Students in London and at the Provincial Associations.

1. A Miller Prize and the James Forrest Medal to Frank Herbert Bullock, B.Sc., Stud. Inst. C.E. (Bristol), for his Paper on "Aerial Ropeways."
2. A Miller Prize to James Gordon Mitchell, B.A., Stud. Inst. C.E. (Manchester), for his Paper on "The Locomotive Cylinder in Design, Manufacture and Maintenance."
3. A Miller Prize to Archibald George McDonald, Stud. Inst. C.E. (London), for his Paper on "The Structure of Steel."
4. A Miller Prize to Harry Wolf, Stud. Inst. C.E. (Manchester), for his Paper on "Civil Engineering under the Roman Empire."

Bayliss Prize.

Bayliss Prizes, awarded on the results of the October and April Examinations, 1921–1922 respectively, to George Thomas Carpenter, M.C., and William Melville jointly, and Alexander Allan Findlay, Stud. Inst. C.E. In connection with the April, 1922, Examination, the Council find that William Herbert Maitland merits honourable mention.

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[1] Has previously received a Miller Prize.

[2] Has previously received a Telford Premium.

[Other Awards.

OTHER AWARDS MADE DURING SESSION 1921–22.

For Papers Printed in Section II of the Proceedings.

Session 1918–1919.

1. A Telford Gold Medal to Professor Ernest George Coker,[1] M.A., D.Sc., F.R.S., M. Inst. C.E., for his Paper on "Photo-Elastic Measurements of the Stress Distribution in Tension Members used in the Testing of Materials."
2. A Crampton Prize to Frederick Francis Percival Bisacre, O.B.E., M.A., B.Sc., Assoc. M. Inst. C.E., for his Paper on "Overhead Track Construction for Direct-Current Electric Railways."
3. A Manby Premium to Ernest Herbert Lloyd, O.B.E., M. Inst. C.E., for his Paper on "Work in connection with the Suez Canal Defences in 1916, which was undertaken by the Egyptian Ministry of Public Works Officials for and in conjunction with the Royal Engineers."
4. A Telford Premium to Rollo Appleyard,[1] O.B.E., M. Inst. C.E., for his Paper on "The Relation between the 'Bight,' 'Span,' and 'Dip' of Catenaries."

Session 1919–1920.

1. A Crampton Prize to Professor Frederick Charles Lea,[1] O.B.E., D.Sc., M. Inst. C.E., for his Paper on "The Effect of Temperature on the Modulus of Elasticity and Other Properties of Metals."
2. A Telford Premium to Alexander Robert Horne, O.B.E., B.Sc., Assoc. M. Inst. C.E., for his Paper on "Strength and Other Properties of Scots Pine."
3. A Telford Premium to Sidney Blencowe, Assoc. M. Inst. C.E., for his Paper on "Railway Location."
4. A Telford Premium to William John Walker, B.Sc., Assoc. M. Inst. C.E., and R. Koivulehto for their joint Paper on "Experiments on Water-Injection in a Gas-Engine and its Effect upon Heat-Distribution."

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[1] Has previously received a Telford Premium.

Session 1920–1921.

1. A George Stephenson Gold Medal to John Henderson Taylor,[1] Assoc. M. Inst. C.E., for his Paper on "The Application of Electric Traction to the Suburban Lines of the Central Argentine Railway."
2. Telford Premiums to Frank Harvey Hummel, M.Sc., Assoc. M. Inst. C.E., and E. J. Finnan, M.Sc., for their joint Paper on "The Distribution of Pressure on Surfaces supporting a Mass of Granular Material."
3. A Telford Premium to Herbert Chatley, D.Sc., Assoc. M. Inst. C.E., for his Paper on "Silt."
4. A Trevithick Premium to George Ernest Lillie, F.C.H., M. Inst. C.E., for his Paper on "Permanent Way on Mountain Railways."

For Papers Read Before Meetings of Students in London and at the Provincial Associations.

Session 1919–1920.

1. The James Forrest Medal and a Miller Prize to Richard Douglas Gauld, M. Eng., Stud. Inst. C.E. (Manchester), for his Paper on "Some Problems in the Maintenance of Railway Permanent Way."
2. The James Prescott Joule Medal and a Miller Prize to Joseph Edgar Dumbleton, Stud. Inst. C.E. (Birmingham), for his Paper on "Aerial Navigation."
3. A Miller Prize to Henry Fowler, Jun., Stud. Inst. C.E. (Manchester), for his Paper on "The Repair of 18-Pounder Cartridge Cases."
4. A Miller Prize to Guy Howard Humphreys, B.A., Stud. Inst. C.E. (Manchester), for his Paper on "Cement Manufacture and Testing."
5. A Miller Prize to Charles Maurice Brain, Stud. Inst. C.E. (London), for his Paper on "Compression Refrigeration."
6. A Miller Prize to Kelvin Tallent Spencer, M.C., Stud. Inst. C.E. (London), for his Paper on "German Float Bridges."

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[1] Has previously received a Miller Prize.

SESSION 1920–21.

1. A Miller Prize to John Thomas Chalk, B.Sc. (Eng.) (London), for his Paper on "Continuous Beams."
2. A Miller Prize to Eustace Alfred Phillipson, Stud. Inst. C.E. (London), for his Paper on "The Increased Efficiency of the Locomotive."

ORIGINAL COMMUNICATIONS

RECEIVED BETWEEN 1 APRIL, 1922, AND 31 MARCH, 1923.

AUTHORS.

Addison, H. No. 4,455.—Mechanical Screens for Circulating Water and Sewage. With 2 sheets of diagrams and an abstract.

Appleyard, R. No. 4,464.—Catenary Measurements. With 3 diagrams and an abstract.

Blackadder, W. No. 4,435.—Practical Formulas for the Delivery of Cross-connected Pressure Mains. With 8 diagrams and an abstract.

Brown, E. O. F. No. 4,432.—Underground Waters in the Kent Coal-field and their Incidence in Mining Development. With 16 tracings and an abstract.

Clark, F. No. 4,447.—The Construction of two Rolling Lift Bridges at Chatham Dockyard. With 8 photographs, 6 prints, and an abstract.

Coleman, G. S., and Smith, D. No. 4,462.—The Discharging-Capacity of Side Weirs. With 26 sheets of prints and an abstract.

Ferguson, C. No. 4,442.—A Gyrostatic Theory of the Ball Bearing. With 8 diagrams.

Ghaleb, K. O. No. 4,439.—Silt-Deposit in Irrigation-Canals in Egypt, and its Prevention. With 3 diagrams.

Gibbs, A. R. See Tudsbery, H. T.

Grantham, R. F. No. 4,465.—The Effect of Groyning upon some Parts of the English Coast. With 3 prints and an abstract.

Gregory, W. No. 4,436.—Westmill Bridge. With 3 photographs and 3 diagrams.

Gyde, C. J. No. 4,440.—Some Bridge-Foundations of Moderate Depth. With 13 photographs, 6 sheets of tracings, and an abstract.

Hall, W. B. No. 4,467.—Inchinnan Opening Bridge. With 3 diagrams and an abstract.

Hearn, G. R. No. 4,452.—The Effect of Shape of Catchment on Flood-Discharge. With 2 prints, a tracing and an abstract.

Hiley, A. No. 4,461.—A Vibratory Process for making Concrete Piles in Situ. With 3 photographs, 8 sheets of prints, and an abstract.

AUTHORS.

Hodge, H. F. No. 4451.—The Removal of Fishy Taste and Smell from Water by the Application of Copper Sulphate. With 1 diagram and an abstract.

Houghton, T. E. No. 4,470.—Evaporation by the Vapour Compression Method. With 15 sheets of diagrams and an abstract.

Howarth, A. H. No. 4,454.—The Effect of Tidal and Upland Waters in the Channel of the River Ribble (Port of Preston). With 2 drawings.

Hurst, H. E., and Watt, D. A. F. No. 4,450.—The Similarity of Motion through Sluices and through Scale Models. With 13 tracings and an abstract.

Juell, B. No. 4,449.—Economical Survey for Submarine Cable. With 2 tracings.

Laidlaw, D. H. No. 4,444.—Reconnaissance Surveys in Trengganu.

Lawrence, J. No. 4,448.—Notes on Aeroplane Surveying and Contouring. With 3 sheets of diagrams.

Lea, F. C. and Stradling, R. E. No. 4,431.—Some Experiments on the Effect of High Sulphur Contents in Portland Cement. With 2 diagrams, 2 photographs and an abstract.

Lloyd-Davies, D. E. No. 4,457.—The Works for the Augmentation of the Supply of Water to the City of Capetown, South Africa. With 7 sheets of tracings.

—————— No. 4,459.—Main Drainage of the Southern Suburbs of the City of Capetown, South Africa. With 7 tracings and 3 prints.

Marwood, H. C. No. 4,456.—Construction of a Concrete Railway-Viaduct in Ceylon. With 2 tracings.

Mead, A. D. No. 4,429.—Solution of a Hydraulic Problem in Sewer Design.

Meares, J. W. No. 4,445.—An Automatic Integrating "Intensity" Rain-Gauge, without Clockwork. With 3 tracings, 3 duplicate prints, and an abstract.

Nolan, T. R. No. 4,438.—Slips and Washouts on the Hill Section of the Assam Bengal Railway. With 28 photographs, 14 tracings, 1 map, and an abstract

O'Shaughnessy, F. R. No. 4,463.—Some Physical Laws and Modern Methods of Sewage Treatment. With an abstract.

Pollock, J. No. 4,434.—Long Tunnel Construction in Rock. With 2 drawings and an abstract.

Radford, W. N. No. 4,460.—The Pollution of Rivers and Streams.

AUTHORS.

Redmond, L. G. No. 4,433.—Reinforced Concrete in India. With 11 photographs, 2 drawings, 9 tracings, and an abstract.

Rice-Oxley, M. K. No. 4,468.—The Cubic Transition Railroad Spiral. With 2 sheets of diagrams and an abstract.

Richards, B. D. No. 4,427.—Development of Water Power in India, with some Considerations in the Design of Large Projects. With 3 diagrams and an abstract.

Richards, H. W. H. No. 4,441.—Twelve Years' Operation of Electric Traction on the London Brighton and South Coast Railway. With 24 photographs, 27 prints, 3 specimen stock cards, and an abstract.

Rounthwaite, J. M. No. 4,453.—The Royal Naval Air Works, Cardington, Bedford, with some notes on their Construction. With a book of photographs, 4 sheets of prints, and an abstract.

Sadler, Ralph. No. 4,430.—The Design of Tall Chimneys.

Smith, D. See Coleman, G. S.

Stoney, E. W. No. 4,437.—Description of Slips on the Nilgiri Mountain-Railway, and Failure of the Glenmore Retaining-Wall. With 7 photographs and 6 tracings.

Stotherd, C. E. No. 4,469.—Ramgarh and Dhil Nadi Irrigation Works, Jaipur State, Rajputana. With 4 tracings and 8 photographs.

Stradling, R. E. See Lea, F. C.

Taylor, W. T. No. 4,466.—Graphical Calculation of Overhead Electric Power Transmission Lines. With 1 tracing and an abstract.

Tudsbery, H. T., and Gibbs, A. R. No. 4,458.—An account of an Examination of the Menai Suspension Bridge. With 5 sheets of prints and an abstract.

Varvill, M. N. No. 4,446.—Description of a Simple Device for arresting Rail-Creep. With 1 print.

Walker, R. D. No. 4,428.—Underdrainage, and its Application to Railway Work. With 6 diagrams.

Watt, D. A. F. See Hurst, H. E.

EXTRA MEETING.

4 May, 1923.

WILLIAM HENRY MAW, LL.D., President,
in the Chair.

THE "JAMES FORREST" LECTURE, 1923.

The PRESIDENT said the members had met that evening, on the Thirtieth Anniversary of the delivery of the first James Forrest Lecture, to listen to a lecture which had been prepared by Sir Richard Glazebrook. It was the ordinary task of a Chairman on such an occasion to introduce the lecturer, but Sir Richard Glazebrook was so well known to all members that such an introduction was in the present instance totally unnecessary. He would therefore at once ask Sir Richard to deliver his lecture, which they were all anxious to hear.

The following lecture was then delivered :—

"The Interdependence of Abstract Science and Engineering."

By Sir RICHARD TETLEY GLAZEBROOK, K.C.B., M.A., D.Sc.,
F.R.S., Hon. M. Inst. C.E.

I WISH in the first place to express my cordial thanks to the Council for the honour conferred on me by the invitation to deliver this lecture. The Secretary wrote, telling me of the subject of the first lecture delivered by Sir William Anderson 30 years ago, "The Interdependence of Abstract Science and Engineering," and stating that "the President feels that this would be a good opportunity to deal with scientific research in relation to engineering practice, and especially with the organization, methods and application of engineering research." I welcomed the opportunity. Some 20 happy years of life had been spent in the attempt to apply science to industry, chiefly engineering industry; and the thought of being able to speak on such a subject in this hall was an alluring one. I accepted gladly, but without reading Sir William Anderson's lecture; when I studied it, I was appalled at the task lightly undertaken. His lecture covers an immense extent; it rests on the experience gained in a long and active life devoted to his profession; it is illustrated by reference to questions and difficulties outside my ken; it is full of knowledge obtained by the study of many problems in science and economics and by the application to these of an acute and critical mind. I cannot hope to treat the subject in a manner which will bear comparison with the skill and ability applied to it by my predecessor. Still, I will do my best: let me commence by quoting the conclusion of his lecture:—

"I have endeavoured to show," he said, "how the history of abstract science, by which I intend to designate the history of researches entered into for the sole purpose of acquiring knowledge of the operation of Nature and her laws, without any thought of reward or expectation of pecuniary advantage, has had its reflex in the records of the engineering profession, and how the most recondite investigations, apparently unlikely to have any direct influence on our practice have, in course of time, become of cardinal importance. I have also ventured to point out how in these days the engineer must banish from his mind the idea that anything can be too small or too trifling to deserve his attention. 'Nothing is too small for the great man' is, I am told, written over the cottage once occupied by Peter the Great at Saardam. The truth embodied in that legend should ever dwell in our minds, for success, I am persuaded, lies largely in close attention to details."

Or, again, referring to the views of Mr. Forrest himself:—"You cannot touch anything in natural science which will not, sooner or later, prove of advantage to the members of this Institution, and which the engineer will not, in time, turn to the moral as well as to the material advantage of the human race."

The experience of the last 30 years has all tended to confirm these views. It is my task to-night to give some few illustrations of the truth of the principles advocated by Sir William Anderson, and to trace some of the effects those principles have had on engineering practice. This effect has shown itself in many forms.

The education of the engineer has been greatly modified in consequence. Institutions devoted to the application of science to industry have grown and multiplied. The State has recognized something of its responsibility; and private persons, public men, and industrial associations have given many proofs that now their importance has been, at least in part, realized. As one of the most striking of these proofs let me mention the recent generous gift to the Royal Society from Sir Alfred Yarrow, a gift intended for the advancement of abstract science—natural knowledge—to use the phrase by which it is known in the Charter of the Society.

By establishing the William Froude tank at the National Physical Laboratory, and by the researches which led to the water-tube boiler and the torpedo-boat destroyer, Sir Alfred had already shown his appreciation of the value of science. May I, as a scientific worker, rather than as an engineer, thank him for all he has done.

But before referring in detail to some of the modern applications of abstract science to engineering, let me briefly consider these various connecting links. And first education. Sir William Anderson, after alluding to the importance attached to the scientific education of an engineer by this Institution, deplores the fact that "except in the noble endowments of the City and Guilds Schools and the Government institutions at South Kensington," in London the movement to secure the necessary training "languishes for want of adequate support." What are the conditions at present? The South Kensington Institutions and the City Guilds College now form the Imperial College of Science and Technology, and the Registrar of the City and Guilds College—the Engineering Department of the Imperial College—has kindly furnished me with some details. The college opened in 1886 with 35 students, of whom 8 were reading chemistry. By 1893 the numbers had increased to 205, including 10 chemistry students and 32 special and post-graduate men. In 1911 the chemistry students were transferred to the Chemical Department of the College. At the end of the last complete session (July, 1922) there were 492 engineering students in the college, including 60 special and post-graduate men, while 138 students of the Royal College of Science and the Royal School of Mines were also receiving instruction; thus the total number was 630. During the period under review 563 students have taken the Internal B.Sc. Degree in Engineering of the University of London, and of these 309 have graduated in Honours. In addition, a certain number have obtained the External Degree.

The Academic Registrar of the University of London has been kind enough to send me the following statistics as to the B.Sc. (Eng.) Degrees of the University since its reconstitution in 1900 :—

	Internal.		External.	
	Pass.	Honours.	Pass.	Honours.
1903	0	5	4	1
1922	168	74	89	39
Totals 1903–22 .	672	622	509	247

Of the above totals 51 Internal Degrees and 25 External were War Degrees. The figures[1] show the immense growth since 1903.

As to the rest of London, conditions have altered since 1893. The following summary supplied by Sir R. Blair, the Educational Officer of the London County Council, gives details for 1913–14 and 1921–22 :—

TABLE I.

ENGINEERING STUDENTS IN LONDON.

	1913-14.		1921-22.[2]	
1. Professional students (day):				
University institutions	718		981	
Polytechnics	506	1,224	725	1,706
2. Evening and part time day . . .		6,580		8,863
3. Day trade schools		981		1,021
4. Other evening institutes		1,521		2,556
Total students		10,306		14,146

The numbers show a great improvement on the condition at the date of the first James Forrest lecture. When, however, we realize that before the war there were some 175,000 persons employed in engineering trades in London, of whom about 50,000 were under 25 years of age, while the number of persons dependent on engineering industries—workers and families—was in 1916 about 750,000, they seem small enough.

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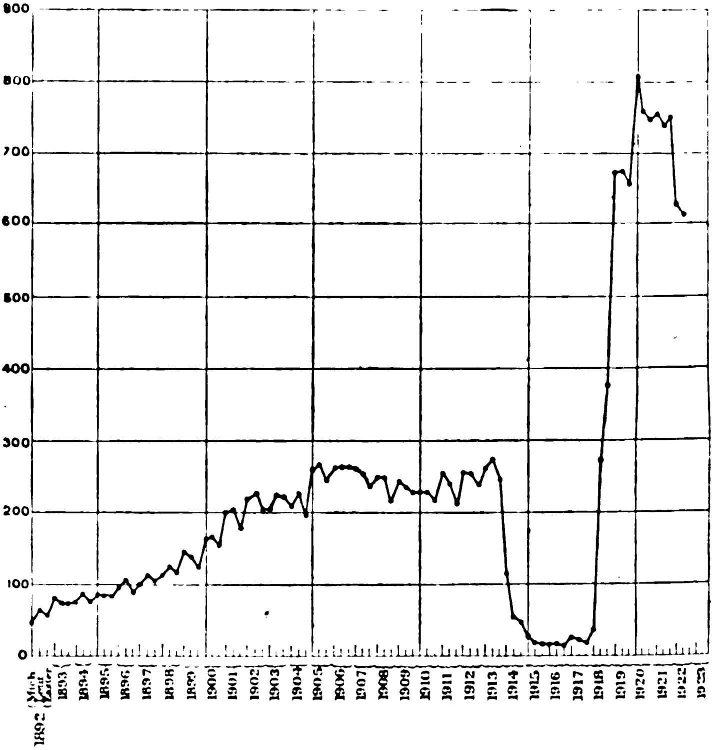
[1] These figures include the degrees of Imperial College students already mentioned.

[2] These members are somewhat increased owing to facilities offered for discharged officers.

The numbers of students in some other engineering centres may also be given.

In pre-war days the Royal Technical College, Glasgow, had 238 day students and 1,994 evening engineering students. Affiliated to it were institutions with 9,820 engineering students. At the University of Manchester there were 57 students in 1893; the number at present is 148. A limit of 150 is fixed by the size of the laboratories. The Manchester Technical College began work in 1906, with 5 candidates for a degree in Mechanical Engineering and 28 for a certificate. In the present Session the numbers are

Fig. 1.



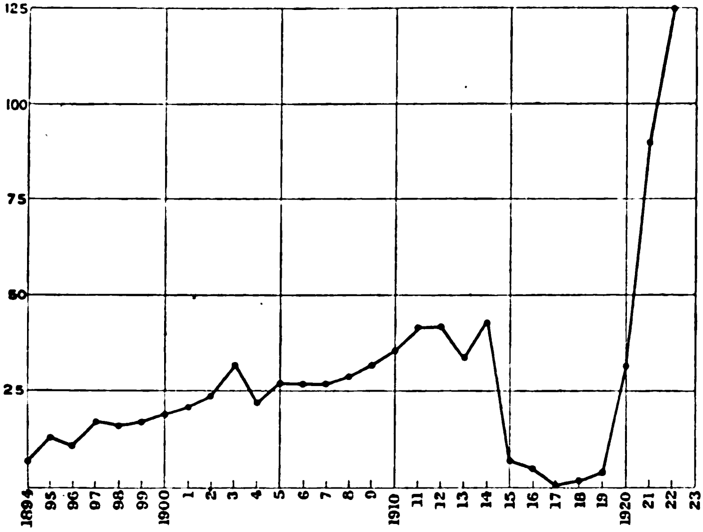
CAMBRIDGE. TOTAL NUMBER OF STUDENTS ATTENDING ENGINEERING DEPARTMENT.

71 and 7 respectively; again, in Electrical Engineering, while in the first Session there were 16 degrees and 40 certificate candidates, in 1922–23 the numbers were 87 and 8 respectively.

Figs. 1 and *2*, which I owe to the kindness of Professor Inglis, show the growth of the Engineering School at Cambridge since 1894. In that year there were some 50 students in the school, and 7 Honours Graduates. In 1920 the students numbered 800, and at present[1] are rather over 600, while the number graduating in Honours in 1922 was 125.

Professor Inglis writes:—"There is no doubt that employers in all directions are attaching increased importance to a university training. Some firms go so far as to say they will not take a man on their staff unless he possesses a university degree. This attitude is growing steadily, and it has become accelerated since the war. The general demand is for a youngster who has had a good all-round training."

Fig. 2.



CAMBRIDGE. STUDENTS GAINING HONOURS IN THE MECHANICAL SCIENCES TRIPOS.

In America there is more than one college with from 1,250 to 1,500 students. At Charlottenberg before the war the number was 1,500.

The educational scheme in London covers a wide range. In the university institutions a large proportion of the students are reading for the B.Sc. degree; in the polytechnics the proportion is, of course, less—a diploma in engineering is the more usual aim.

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[1] The fall is due to the absence of service students.

In the report already alluded to, Sir R. Blair points out that it is generally recognized "That in addition to the course of more strictly engineering subjects—wide enough in itself—the professional students of engineering must have an acquaintance with modern languages and with economics, including questions of cost of production and of management of labour."

Of the evening classes some are intended for men who hope to proceed to a university degree, but cannot afford whole-time study, and in these an advanced stage of instruction is reached. Others again aim at fitting for their work large numbers of foremen and skilled artificers. The technical training of workmen in the engineering trade is carried on mostly in evening classes; in many of these mathematics and drawing, together with, in some cases, chemistry and physics, have a prominent place.

Part-time classes for apprentices have been arranged in connection with various large works; perhaps the most complete scheme is that run at the Woolwich Polytechnic for the trade lad at the Royal Arsenal. It includes mathematics, mechanics, engineering drawing, electricity and magnetism, and chemistry, and the work done is considered along with the works reports when dealing with promotion. Another such scheme is that of the Gas Light and Coke Co. for gas-fitter apprentices, described in a Paper by Mr. Goodenough, read before The Institution of Gas Engineers in 1913, entitled "Creating an Outdoor Staff." Up to the present 330 trained under the scheme have entered the shop, about 30 have been promoted to the staff, and 30 are now in training. In this course, and in the apprenticeship schools established by the Council in various localities, in addition to the subjects mentioned above, English forms a prominent subject of instruction. Thus, while I would press strongly the hope that the efforts made since 1893 to extend the knowledge of the fundamental principles of engineering should in no way be relaxed, the advance has been very considerable. A serious attempt has been made to impart more "than the slender amount of theoretical knowledge," asked for by Sir William Anderson, "to the various grades of employment," and the attempt has, I claim, met with success. As evidence of this, let me quote a few words from a report by Mr. M. F. Ryan, Director of Munition Gauges, Ministry of Munitions, to whose great organizing skill and tactful handling of every source of supply the country owes a great debt. Dealing with the question of gauges he writes:—"The success of the L.C.C. in their supply organization was greater than that of any of the other training sections in the country." . . . . "The national asset underlying this success is a body of men possessing a high degree of manual skill in addition to well-educated minds and capable of transmitting their skill and knowledge to others."

Possession of "a high degree of manual skill" is no less necessary than high academic training, for without works experience no student can become an engineer, and the unanimous conclusion of the committee of this Institution, which some years since reported on the training of an engineer, that "Engineering training must include several years of practical work as well as a proper academic training," nowadays perhaps needs specially enforcing.

Nor is the view that science must go hand in hand with practice confined to the heads of the engineering profession only. At the Trades' Union Congress at Southport in 1922 a report on apprenticeship was presented from the shipbuilding, engineering, iron and steel, and building group of the General Council. After referring to the difficulty found in many large areas in obtaining instruction other than that provided in the workshop, especially in cases where part-time training is required during the day, the report continues: "It is now generally accepted that if industry and commerce are to progress, the workers engaged therein must be allowed to exercise their intelligence," and suggests the following grading for apprenticeships :—

(1) The trade school, providing for workshop training and constituting a selecting ground for testing the capabilities of the apprentice.

(2) The technical school where instruction of a scientific and theoretical nature would be given.

(3) The universities or higher technical colleges for supplementary training of apprentices of proved special ability.

This, it will be noticed, is practically the scheme in existence in London under the County Council.

Again, several of the Joint Industrial Councils—the Whitley Councils—have given special consideration to problems of education and research, and have appointed committees to deal with them. The report establishing the Councils specifically mentions "industrial research and the full utilization of its results" as one of their objects. Prominent among such are the Pottery and Building Trade Councils. A sub-committee of the Pottery Council dealing with Research, Inventions, and Design recently recommended the establishment of a £50 scholarship.

Some of the trades unions, notably the Association of Engineering and Shipbuilding Draughtsmen and the Electrical Trades Union, have shown by articles in their journals their appreciation of the importance of science as an element in progress, while others have taken great interest in the applications to workers of modern psychological research; in a recent article Mr. W. Graham, M.P., emphasized the importance for the world of the possibility which trade unions have before them in "the scientific study of what is not very happily termed industrial fatigue."[1]

In reply to an inquiry as to the attitude of the workers towards science in relation to industry the secretary of the Iron and Steel Trades Confederation writes, after a reference to the appreciation shown by the men of classes provided by local education authorities, "The old-fashioned workman who was content to plod along by rule of thumb, and who had no desire to learn anything of the processes which gave him employment, is disappearing, and even that type of man is so far aware of his own deficiencies that he is making tremendous sacrifices so that his sons can have a scientific education. This is evidenced by the number of applications for places in secondary and technical schools, the applications far and away outnumbering the places available."

Again the Assistant General Secretary of the Union of Post Office Workers writes thus :—

> "So far as it is possible for me to speak in general terms on behalf of our members, I should say that we as an organization are keenly alive to the interdependence of Science and Industry.
>
> "Our attitude as a Union towards science and invention is not one of resistance, but we hold the view very strongly that the social advantage resulting from an improved process should be, to some extent at least, secured for the alleviation of the lot of the industrial worker.
>
> "To-day, scientific development usually carries with it a simplification of process and a reduction in the wages standard. In our view the gain due to the introduction of improved machinery should be devoted in part to the improvement of working conditions."

For my part I am glad to take this opportunity of expressing my own agreement. It is, in my opinion, an essential condition of progress, and, as scientific men, we have no desire to deny the workers their fair share in the results due to our advance.

Another matter of great importance is the supply of trained workers who will bring to their duties in factories and workshops a knowledge of scientific principles and an acquaintance with scientific methods.

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[1] I owe much of this information to the courtesy of Mrs. Wootton, of the Research and Information Department of the Trades Union Congress.

The scheme of industrial bursaries started in 1911 by the Royal Commission for the Exhibition of 1851 has this for its object. One of the difficulties which confront a student from a university or technical college who desires to become an engineer arises from the small pay of an apprentice. At first he is worth but little to his firm, and his pay is in proportion to his worth. The Commissioners offer each year, to a large number of university institutions, nominations to bursaries to be awarded to students who have done well in some branch of science and who propose to go into works. Each student before receiving his emolument must be placed in some position approved by the Commissioners, and the amount of the bursary depends on his means and on the pay he receives from the works; it is the intention of the Commission to bring this up to a living wage. At present the nominal sum is £150, which may at the discretion of the Commission be raised to £200; the actual sum varies with the other conditions of the applicant. The bursary usually lasts for 2 years, at the end of which it is supposed the worker should be in a position to support himself. The scheme was inaugurated in 1911. In the following 3 years sixty-three graduates of various universities were enabled to enter the engineering profession, bringing with them the scientific knowledge acquired during three years' successful academic training. The scheme was suspended during the war, but is now in active operation again; and up to Dec. 31 last about £19,000 had been expended in the assistance of 185 bursars. The complete list is most interesting reading; without this help few, if any, of the candidates could have become engineers; their parents' income varied from £75 a year up to £500 or £600. Quite a considerable proportion now hold responsible and well-paid posts.

The scholarships established years ago by Sir Joseph Whitworth are almost too well known to need special reference. They operate in the opposite direction to the bursaries scheme, in that they offer opportunities of a college training to men with works experience.

Among the agencies doing most valuable work in the introduction of men trained in science and literature to the engineering and other industries must be reckoned the appointments boards of the universities. The Cambridge University Appointments Board[1] was established in 1902, at first in a tentative manner; it became a permanent Board in 1906, and since that date has been increasingly active. It consists of representatives of the university and of industry, and has throughout received great assistance from the engineers and men well known in the business world who have voluntarily placed their services at the disposal of the university.

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[1] An interesting account of the pre-war work of the Board will be found in a paper by the secretary, Mr. H. A. Roberts, on "Education in Science as a preparation for Industrial Work," Royal Society of Arts, March, 1912. Reference may also be made to the Reports of the Board published annually in the Cambridge University Reporter.

According to its last report :—

"In the year ended Dec. 31, 1922, 373 appointments were obtained on the introduction of the Appointments Board. The appointments were distributed as follows :—

Government Departments	13
Administrative appointments in commerce and industry	94
Manufacturing and technical appointments	87
Colonial administration	16
Secretaryships	2
Agriculture and forestry	16
Indian and Colonial railways	12
Public Works Department and Surveys	7
Railway traffic	3
Journalism and publicity	5
Publishing	3
Articled clerkships	3
Museums	3
Miscellaneous	1
Educational appointments	120"

Men who intend to take up the engineering profession usually obtain their degree in honours in the mechanical sciences tripos, and of these honours graduates the following are lists for the past 3 years :—

MECHANICAL SCIENCES TRIPOS, CAMBRIDGE, 1920-1923.

1920. Number of Graduates, 30.

Director or partner in manufacturing concern	1
Subordinate positions in manufacturing firms (ranging from draughtsman or works-managers' assistant up to works manager	15
Assistants to consulting engineers	3
Assistant teacher	1
In service of Government, municipalities, or railway companies in the British Isles	5
Miscellaneous	1

(Four occupations unknown.)

1921. Number of Graduates, 92.

Pupils or apprentices	38
Salaried posts (Government commercial, Government engineering services abroad, teaching, patent agents)	42
Employment other than engineering	1
Family business	8

(No record of three persons.)

1922. Number of Graduates, 125.

Pupils and apprentices	37
Salaried posts	38
Still at Cambridge	9
Obtained own opening	30
No employment	9

In connection with the work the Secretary writes :—

"In industry we are continually and increasingly being asked for men, if not highly-trained research men, at any rate sufficiently grounded in the elements and terminology of science, to consider the problems of production from a scientific point of view. I have before me to-day (Mar. 26, 1923) a letter from a prominent telephone company asking for such a man ; last year by far the largest manufacturers of hosiery in the world were looking for a first-class physical chemist to study the question of shrinkage ; the Calico Printers' Association and the large cotton firms generally are concerned to investigate the scientific reasons as to why a certain kind of sizing is necessary to get the best results when dyeing. A large cocoa firm, to name only one other from a very large number of similar inquirers, were seeking a scientifically-trained man to investigate the chemistry of the natural fermentation of the bean within its pod, the whole flavour and working state of the bean depending on the best moment of arresting this process. Nor must one omit to mention the research being carried out by the large oil companies both on their own and through us in Cambridge, and by individual workers of distinction such as Ricardo at Shoreham."

These instances are, it is true, not all engineering, but enough has perhaps been said to indicate the close interdependence between exact science and all forms of industry ; and the evidence of other similar boards might be quoted to the same purpose. Turning next to institutions and associations designed to promote the application of science to industry, it is not improper, I trust, for me to place first the National Physical Laboratory, and I do this the more readily, speaking in this hall and to this audience, because of the intimate connection which has existed between The Institution and the National Physical Laboratory since its foundation. Your representatives on the General Board of the Laboratory have been your Past-Presidents—Sir John Wolfe Barry, Sir William Preece, Sir William White, Professor Unwin, and Sir Robert Elliott-Cooper. Nor are these all the members of your Council who have given unstinted help. Dr. Maw has served for long on the General Board and Executive Committee; Colonel Crompton, ever young, is almost a perpetual member, and his enthusiasm has inspired and supported many of our most interesting undertakings. Sir Benjamin Baker, Sir Andrew Noble, Sir Frederick Donaldson, Sir Maurice Fitzmaurice, Captain Sankey, Sir Henry Oram, Sir John Aspinall, Sir Archibald Denny, Sir Robert Hadfield, Mr. Alexander Siemens, Sir John Snell, Sir Alfred Yarrow, Sir Charles Parsons, Sir Dugald Clerk and Sir William Ellis are some of those to whose assistance success is due. Mr. Basil Mott, the designer of the Froude tank, has helped our work in a marked degree.

I am asked to deal with the organization, methods, and application of engineering research. Some of you have probably seen the latest Report of the National Physical Laboratory, a document of over 200 pages submitted to the General Board some 6 weeks since. On looking through it, I felt that the easiest way to discharge the duty laid on me by the Council would be to read extracts from that Report. I do not propose to take that course, though I shall make use of it when dealing later with some of the recent applications of science to engineering. For the present let me refer briefly to the history of the Laboratory, its work, and its organization. In 1893 the Laboratory was not: it existed only in the hopes and imagination of a few enthusiasts. At the British Association at Cardiff in 1891 Sir Oliver Lodge, in his address as President of Section A, had referred to the need, and outlined some of the work for the Laboratory. "The further progress," he said, "of physical science in the somewhat haphazard and amateur fashion in which it has been hitherto pursued in this country is becoming increasingly difficult; the quantitative portion especially should be undertaken in a permanent and publicly-supported laboratory on a large scale."

The seed germinated slowly: the late Sir Douglas Galton, in his presidential address at Ipswich in 1895, pressed the same view, contrasting our position with that in Germany. A British Association committee was set up, with Lord Rayleigh as Chairman, which reported in 1896. Lodge was the Secretary. Its unanimous conclusion was that—"If England is to keep pace with other countries in scientific progress, it is essential that such an Institution should be provided, and this can scarcely be maintained continuously on an adequate scale except as a National Laboratory, supported mainly by Government." Lord Salisbury appointed a Treasury Committee, representative of science and industry; and its report that a public institution should be founded for standardizing and verifying instruments, for testing materials, and for the determination of physical constants was approved by the Government. The Committee had examined more than thirty witnesses, and their evidence was conclusive as to the need of close co-operation between science and industry. The interdependence of the two was recognized. One American writer, speaking of the growth of German industry, assigned as a principal cause the perfected alliance between science and commerce existing in that country. "Science," he says, "there no longer seeks court and cloister, but is in open alliance with commerce and industry"; and in a lecture given before the Royal Institution shortly after I wrote: "It is our aim to promote this alliance in England, and for this purpose the National Physical Laboratory has been founded."

The scheme of organization was admirably adapted for its ends. The ultimate control was vested in the President and Council of the Royal Society, who were to act through a General Board and an Executive Committee. The Board consists of certain official members, six in number, and thirty-six ordinary members appointed by the Council of the Royal Society. Of these twelve are nominated by the six leading technical societies[1]; the remaining twenty-four are selected by the Council from men of science and leaders in industry and trade, who need not be—many indeed are not—Fellows of the Society. The Executive Committee of twelve, with certain officials, is appointed from the Board, with the proviso that half the ordinary members must be selected from the representatives of the technical societies.

The Executive Committee was to have the immediate management of the Laboratory, and was responsible for its finance. The Director was responsible to and took instructions from the Executive Committee, but, subject to such instructions, had the sole direction and control of the officials of the National Physical Laboratory and of the work done within it. Such was the scheme, and under this scheme the Laboratory worked until 1918. Sir Arthur Rücker and Mr. S. E. Spring Rice, of the Treasury, took the main part in its preparation, and it is a privilege to put on record my testimony to the value of their work.

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[1] These are The Institution of Civil Engineers, The Institution of Mechanica Engineers, The Institution of Electrical Engineers, The Iron and Steel Institute, The Institution of Naval Architects, and The Society of Chemical Industry.

But in time the building spread beyond its foundations. Conditions during and after the war altered; the financial responsibility became too great for the Royal Society to bear.[1] In 1900 the ordinary expenditure was £5,479; by 1913–14 it had grown to £38,003; and since the war it has largely exceeded that amount. The value of the work done for which payment is received now approaches £100,000.

Thus a change was necessary, and in 1918 the Laboratory became a Government institution: the responsibility of the Royal Society ceased. The General Board and the Executive Committee, though appointed by the President and Council, are responsible to the Department of Scientific and Industrial Research; the real control rests in the hands of the officers of the Department and of its Advisory Council.

During the period, 1900–1914, thirteen volumes of scientific researches were published. We began in a small way with two Departments—Physics and Engineering—and a staff at the end of 1901 at Bushy House, of the Director, eight scientific assistants,[2] an accountant, four mechanics, and an engine-driver—fifteen in all. In addition, Kew Observatory, with Dr. Chree and a staff of sixteen, formed the Observatory Department.

According to the Report for 1922 there are now eight departments at Teddington, each with its superintendent or head and staff of scientific assistants, observers, and other workers, in all[3] 484 persons; the buildings consisted of Bushy House and a small workshop; now they constitute a small town. At the start the

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[1] The following figures may be of interest, giving the totals for 14 years:—

Income Account.

	£
Treasury grants to Laboratory	80,500
,, ,, for Aeronautics	20,182
Receipts for work done	166,633
Donations	15,230
	£282,545

Capital Account.

	£
Treasury grant	75,941
Private donations	55,967
Provided out of Income	24,290
	£156,198

[2] These were Dr. Harker, Mr. Albert Campbell, Dr. Carpenter, Mr. Keeling, Mr. F. E. Smith, Dr. Stanton, Mr. Jakeman and Mr. Melsom.

[3] At the end of the war period the staff numbered some 600.

expenditure was small, salaries were low; in fact, one technical journal, commenting on the terms of appointment, anticipated that "the duties will be equally light." As to the organization, in those days it was simple; the Director was able to keep in close touch with everything that went on; the staff worked as a whole, discussing and consulting with each other as difficulties arose. We were a team working with a common object inspired by the belief that the application of science to the industrial problems set before us was of supreme importance, and ready to devote ourselves to that end. That spirit animated the work for 20 years, and on its maintenance depends in great degree the future of the laboratory. It exists still, though under altered conditions, brought about in part by the natural growth of the laboratory, in part by its connection with a Government Department. Freedom from financial troubles has been gained; freedom to develop under the fostering care of the Royal Society is replaced by the task of working under Treasury conditions imposed on the executive and the staff alike.

I propose to return later to the researches carried out in recent years and pass on to some of the other organizations established in late years for the promotion of science in its application to industry. Reference has just been made to the Department of Scientific and Industrial Research, which now forms a most powerful agency connecting science and industry.

A Committee of Council dealing with industrial and scientific research came into existence first under the auspices of the Board of Education, when Mr. Pease was President of the Board. At a later date it was connected with a department of its own. On Dec. 1, 1916, a deputation from the Conjoint Board of Scientific Societies, headed by Sir Joseph Thomson, President of the Royal Society, waited on Lord Crewe in this hall. Sir Joseph Thomson, in his address, dealt with the importance to industry of research in pure science; he was supported by Sir Maurice Fitzmaurice, the President of this Institution, speaking on behalf of engineering, and by Professor H. B. Baker, F.R.S., who dealt with some of the problems of industrial chemistry.

In his reply Lord Crewe, Lord President of the Council, outlined some of the duties and responsibilities of the new Department.

The following is an extract from the official notice issued at the time (Dec. 1, 1916):—

"The Government have decided to establish a separate Department of Scientific and Industrial Research for Great Britain and Ireland under the Lord President of the Council, with the President of the Board of Education as Vice-President. They have also decided, subject to the consent of Parliament, to place a large sum of money at the disposal of the new Department, to be used as a fund for the conduct of research for the benefit of the national industries on a co-operative basis."

The sum of money placed at the disposal of the Department for this co-operative work was £1,000,000, and is held in trust by the Imperial Trust for the Encouragement of Scientific and Industrial Research for the work of the industrial research associations referred to in more detail in another section of this address. But in addition the Department controls other agencies[1] and administers further funds devoted to the application of science to industry. The National Physical Laboratory is considered elsewhere; the Geological Survey and Museum is also one of the activities of the Department; while under the Fuel Research Board and the Food Investigation Board problems of vital interest are being carried out.

Further work of great importance is dealt with by the Co-ordinating Boards for Chemistry, Physical Engineering, and Radio Research established by the direction of the Government to co-ordinate researches which are of value to the fighting services, and which also have industrial applications. The researches are carried out at the National Physical Laboratory or at other research establishments, usually under Government control, though to some extent university laboratories are made use of. In addition to these there are a number of Boards dealing with special problems: the work of one such—the Lubrication Research Committee—is of utmost importance to engineers, and will be referred to again; while finally the Department applies a portion of its funds to grants to research workers and students at the universities. The lists of aided researches and of publications by individuals in receipt of grants, given in the appendixes to the Report, show the wide extent of this branch of its activities and its importance to industry.

Among the many agencies which in recent years have contributed to the application of abstract science to industry the British Engineering Standards Association holds a prominent position.

It was formed—at the instance of Sir John Wolfe Barry, who until his death held the position of chairman—in 1901, when six or eight members were nominated by this Institution to investigate the question of steel sections. The organization now numbers some 2,000 members and over 400 committees. It has issued about 170 specifications, covering a wide field. Standardization is based on accurate measurement, and in the preparation of many of these specifications fundamental principles have been evolved and researches of difficulty and importance have been required.

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[1] Details will be found in the Report of the Committee for Scientific and Industrial Research for 1921-22, published by the Stationery Office.

Another most valuable method of applying science to research is the research laboratory[1] of a large firm, and though of necessity the work of such a laboratory is aimed mainly at improving the products of the firm, the fact is being realized more and more that for this purpose also investigations into pure science are essential. I need hardly do more than refer to the chemical investigations which have formed the basis of the German dye-industry or the researches of Haber and his colleagues on the nitrogen problem—an inquiry advanced here in England by the work of the nitrogen products branch of the Ministry of Munitions.

Perhaps the best known of the engineering research laboratories controlled by an industrial firm is the establishment at Schenectady of the General Electric Company of America. To quote from Dr. Mees's book : "This laboratory is not concerned primarily with the solution of works problems or with investigations on the manufactured problems of the company ; it has deliberately sought entirely new discoveries, new applications of materials, and new developments in the art of electricity.". . . "From it have come the metalized carbon and the drawn-wire tungsten filament lamp, the nitrogen-filled high-efficiency lamp, the magnetite arc-lamp, and the Coolidge X-ray tube." The development of each of these has involved investigations of great importance to pure science, and among the workers at the problem, which of all others is now arousing the interest of the physicist, the constitution of the atom, Dr. Langmuir, of the G. E. C. Laboratory, occupies one of the leading places.

The research section of the Westinghouse Electric and Manufacturing Laboratory is organized somewhat differently. It is divided into sections and embraces work from the purely theoretical side of the problems presented, to the practical application of principles and problems in the factory.

The laboratory of the Eastman Kodak Company affords another instance of the union between pure science and industry ; it is sufficient to refer to the work on light filters which has proceeded from it, and the improvements thereby rendered possible in every branch of photography, and not least in the microphotographic work so important to engineers.

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[1] See "The Organization of Industrial Scientific Research," by Dr. Kenneth Mees, of the Eastman Kodak Research Laboratory.

In England there are few such great laboratories, and for this there are various reasons. In a recent address, in which he stressed the importance of psychology to the worker, Lord Balfour wrote:—"As a nation we have underrated the great work which science can do for industry. We are too apt to think that science is only for men of science, and that learning can interest only the learned." The war has taught us another lesson, and the necessity of close co-operation with science is fully realized in every branch of industry. In a publisher's circular recently received, it is stated that:—"Whatever the commodity, if it be of any considerable importance, the factory producing it without the scientific guidance of a closely-allied laboratory is working blindfold."

But there were pioneers who recognized this long since. Manganese steel was produced in 1882 from the laboratory of Sir Robert Hadfield, in Sheffield, as the result of a scientific inquiry into the properties of alloys; and many results of high value to science have come from the same source. The foundations of photomicrographic research were laid by Sorby in a Sheffield laboratory, and the Brown-Firth Laboratories of John Brown and Sons have for some time past been engaged under Dr. Hatfield on fundamental researches into the properties of alloys of steel.

In Manchester there are important laboratories attached to the Westinghouse works, and electrical engineers know the value of the researches which go on there. But perhaps I may be permitted to refer in more detail to the work which has recently been begun under my friend and valued colleague, Mr. C. C. Paterson, M. Inst. C.E., at the new laboratories of the General Electric Company at Wembley. Many of my audience who were present at the opening ceremony some few weeks back must have gained a new conception of what is meant by a works' laboratory, and of the interdependence of pure science and electrical engineering. Let me quote from Mr. Paterson's own statement of his ideal, as given in his account of the laboratory:—

"The question is sometimes asked whether the laboratories undertake 'pure research' or confine themselves to 'applied research.' It is a fundamental assumption that the question is meaningless. If 'applied research' means the application to practical problems of existing scientific knowledge, then it is a misnomer to call it research at all, though the laboratories may have to assist the works in this way. Every practical problem that cannot be solved by the trained experience of a factory superintendent, that is to say, every problem that is likely to come before the laboratories, requires for its solution, not only new information concerning facts, but also some extension of scientific principles. It is the business of the laboratories to provide that extension, and the results are not the less intellectually interesting because the research for them was dictated originally by some motive other than pure curiosity; nor are the methods by which the search is conducted in any way different from those characteristic of the purest and most academic research."

There is no restriction as to the character of the problems attacked or the methods of attacking them, except that those problems are naturally taken up which seem to offer the best chance of yielding information of use to those branches of the industry in which the company is or may be interested. When, therefore, the laboratory staff is asked to investigate some manufacturing difficulty, they do not restrict their inquiries to the obvious avenues, but attempt to understand more fully than before the fundamental principles upon which this branch of manufacture is based. The line that the research will follow depends quite as much on the interests of the worker as on the practical problem which provided the starting-point. It has happened already, and will certainly happen again, that a research started in the hope of solving one problem has ended in solving another which was not contemplated in the first instance. Furthermore, because most of the problems arise from the manufacture of electrical appliances, it does not follow that work can be confined to the practice of those branches of science which appear most directly connected with these devices. For instance, lamp research is not confined to work in vacuum physics; it is equally concerned with metallurgical research, glass research, radiation from solids and gases, high-tension electric phenomena, and chemistry—particularly reactions at very high temperatures.

A research laboratory is not complete unless it contains members interested in almost every branch of science, and provides facilities for these and also other classes of work. This wide branching of research, starting originally from an apparently restricted problem, has an incidental advantage; it makes a laboratory which is fitted to undertake one kind of industrial research fitted to undertake all. It is true that there is a certain amount of specialized equipment, such as the laboratory factories, which form part of the organization, and which are necessary to bridge the gap which separates research from its successful incorporation in factory production. These are a very important part, because failure at this point may render useless, and often has done so, magnificent efforts on either side of the gap. But they are a small part and, whatever be the needs of the future, the research laboratories can readily direct their efforts to any point from which the new call comes.

These are great aims: a company is to be congratulated which has at its head a man who can make such aims possible, and in its service one who can realize [1] the ideals with which he has inspired his chiefs.

But laboratories such as this are possible only to the favoured few. In the case of any but the richest firms the cost would clearly be too great. While a works' laboratory in which routine tests are applied to the materials used and products manufactured is a necessity even in the case of small firms, these can only secure the advantages of pure research by some form of co-operation; and it is to meet this case that the Research Associations of the Department of Scientific and Industrial Research already alluded to have been evolved.

A number of firms concerned in the same industry combine for research purposes. Each contributes its share to the expenses, and the total sum thus raised is met by a grant by the department from the million fund set aside by the Government for this purpose. In most cases the grant is on the pound-for-pound basis, and is guaranteed under certain conditions for 5 years; there is a limit of £5,000 a year to its amount. The associations are managed by a committee or board, on which the Department is represented; each has a Director of Research as its technical officer; the experimental work is carried out either at a central laboratory run by the Association or in many cases by arrangement with some suitable institution, e.g., The National Physical Laboratory or the local university.

The results of any research carried out by the Association belong as a rule to all the members equally, but a firm may, if the committee approves, have an investigation carried out for its own special benefit at cost price.

Each firm subscribing receives a regular service of summarized technical information, and may obtain a translation of any foreign article in which it is interested. It may put technical questions to the Committee and recommend specific objects for investigation; these, if approved by the Committee, are carried out at the cost of the Association for the common benefit of the members. The firm has also the right to use any patents arising from the investigation, on the payment at most of a nominal fee.

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[1] In this connection the titles of the following list of Papers already published by the staff are interesting :—

LIST OF PAPERS ON SCIENTIFIC SUBJECTS BY THE RESEARCH STAFF G. E. C.

Disappearance of Gas in Electric Discharge. Parts 1–3.
Method for Micro-Analysis of Gas by Pirani, etc.
High Temperature Phenomena of Tungsten Filaments.
Disappearance of Gas. Part 4.
Efficiency of Impurities on Re-crystallization, etc.
Cathode Disintegration.
A Problem in Viscosity. The Thickness of Liquid Films formed on Solid Surfaces under Dynamic Conditions.

The number of associations formed up to the present is twenty-four, and among these those most interesting to engineers are perhaps the Iron Manufacturers' Association, the Association of British Motor and Allied Manufacturers, the Portland Cement Manufacturers' Association, the Non-Ferrous Metals Association, the Scientific Instrument Association, the Electrical and Allied Industries Research Association, the Cast-Iron Research Association. There is no association to deal with general engineering problems.

The last report of the Department of Scientific and Industrial Research deals at some length with the nature of the researches undertaken by associations; it points out that there is little basic difference between the fundamental research work required by industry and "pure" research, and it states that, while in pure research the phenomena investigated and the inclinations of the worker furnish the only directive forces, in industrial research "the work has a distinct purpose in view, which the investigator must bear in mind constantly. He cannot afford to follow attractive by-paths unless he believes they will lead him to a relevant destination." The outlook is possibly somewhat less wide than that taken by Mr. Paterson—a consequence no doubt of the fact that the investigator is working under a board or committee; still the report is able to give some definite examples of the manner in which research into fundamental principles has been required for, and has led to the solution of, an industrial problem.

Thus in one case—not, it is true, an engineering question—three cargoes of apples from Australia valued at from £30,000 to £50,000 were lost from a disease known technically as "brown-heart," said to be due to insect injury in the orchards. Investigation based on a research into the physiology of various foodstuffs proved that the apples had been "suffocated" by the carbonic acid they themselves had produced during the voyage.

Of more direct interest to engineers is the Report on Alloy Steels produced by the Motor Manufacturers' Association, work carried out for the Non-Ferrous Metals Association, and some of the investigations of the Electrical Research Association referred to below. This Association is financed in great measure by the Institution of Electrical Engineers and the British Electrical and

Allied Manufacturers' Association, the Councils of which have shown in many ways their appreciation of the interdependence of science and engineering.

Up to the present, obedient to the President's behest, I have dealt mainly with the organization and methods of engineering research, but my address would be singularly incomplete if I did not attempt an account of some of its recent results and of its marked progress. And here my real difficulties commence. Statistics and the kindness of many friends have made the earlier portion of my task comparatively light.

But now the difficulty is what to select, where to begin. The problems are so numerous in which there is a close connection between theory and practice; they can be found in every branch of engineering, perhaps with more striking effect in electrical and metallurgical science, in the laws of stress and strain in structural materials, and in the fatigue of parts subject to vibration, rather than in the questions which pertain more closely to the domain of civil engineering. Let me deal first, briefly and incompletely it must be, I fear, with that branch of electrical engineering—radio, or wireless telegraphy—which at present exercises such a fascination over the popular mind, which is already and will be to a greater extent in the future a link to bind together all nations of the earth. Sir William Anderson refers to Preece's early experiments between Lavernock and Flatholme, a distance of 8 miles, as a startling consequence of electro-magnetic theory. Now the earth is girdled with a wireless chain depending from two, or at most three, great stations. I have just received from the International Union for Scientific Radio Telegraphy details of a scheme for the determination of longitude in which the principal co-operating stations will be Bordeaux, Annapolis, and Pearl Harbour.

In the year 1865 Clerk Maxwell read before the Royal Society his Paper on "The Equations of the Electro-Magnetic Field." It was an attempt which has stood the test of time—the conditions which led Lorentz and, later, Einstein to introduce certain modifications were not considered by Maxwell—to apply mathematical reasoning to those principles, enunciated by Faraday, on which the construction of generators and motors, transformers, and practically all electrical machinery is based.

This reasoning led him to the result that the effect of changes in an electric current in a conducting wire would be propagated through space with a speed depending on the two constants[1] which define the electric and magnetic conditions of the medium surrounding the wire. The values of these constants for air can be found from electrical considerations, and hence the velocity with which electro-magnetic disturbances are propagated can be calculated.

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[1] The velocity is given by $1/\sqrt{k\mu}$ where k is the inductive capacity and μ the magnetic permeability of the surrounding medium.

To quote his words :—" We now proceed to investigate whether these properties of that which constitutes the electro-magnetic field, deduced from electro-magnetic phenomena alone, are sufficient to explain the propagation of light through the same substance," and his conclusion is " The agreement of the results seems to show that light and magnetism are affections of the same substance and that light is an electro-magnetic disturbance propagated through the field according to electro-magnetic laws."

Maxwell found that, when the calculations were made, the resulting value for the velocity[1] was approximately equal to the velocity of light.

The work was extended in his " Treatise on Electricity and Magnetism," published in 1873. The values of the velocity of light and the velocity of propagation of electro-magnetic waves were not known then with present-day accuracy, and he concludes that they are quantities of the same order of magnitude. A glance at present-day figures shows that they are identical, and the electro-magnetic theory of light is universally accepted. Nor was the result true only for propagation through air or interstellar space; such observations as were then available showed that, in all probability, it held for all transparent media, though there were discrepancies, known now to be due to dispersion, which required explanation. But there was a wide gap between this theoretical deduction of Maxwell and the wireless telegraphy of to-day, which needed many more investigations in " pure " science before the bridge was complete. We at the Cavendish Laboratory—I was a student at the time—implicitly believed in its truth; but no one had received electro-magnetic vibrations—at any rate to his certain knowledge. The method of generating them and the means for measuring them were still to come.

For the former we have to go back to a remarkable Paper[2] by a very distinguished Honorary Member of this Institution, Lord

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[1] Messrs. Rosa and Dorsey of the Bureau of Standards, discussing the various determinations of the electro-magnetic velocity, express the view that the figure $2 \cdot 9980 \times 10^{10}$ cm./sec. is accurate to 1 part in 10,000, while the best results for the velocity of light is, to the same accuracy of measurement, $2 \cdot 9986 \times 10^{10}$ cm./sec. " v ". See " Dictionary of Applied Physics," vol. ii.

[2] *Phil. Mag.*, 1855.

Kelvin. Helmholtz[1] seems to have been the first to conceive that the discharge of a condenser through a wire might consist of a forward and backward motion of electricity between the coatings—a series of currents in opposite directions. Sir William Thomson took up the question mathematically and investigated the phenomena. He showed that under certain conditions there would be oscillations of periodic time $2\pi\sqrt{LC}$, where L is the inductance of the coil, and C the capacity of the condenser. These oscillations must, according to the theory, give rise to waves travelling out into space with the electro-magnetic velocity. Fitzgerald, at a meeting of the British Association, had predicted in 1883 that they might be produced by utilizing the oscillatory discharge of a Leyden jar, and Sir Oliver Lodge in 1887 produced and detected them. For their detection the principle of resonance was employed. Any mechanical system free to vibrate has its own period of oscillation, and the application to it of a series of small impulses, at intervals consonant with the free period of the system, results in a disturbance of large amplitude; so, too, an electric system having capacity and inductance has its own period of electrical oscillation, and, if this coincides with the period of incoming electrical waves, electrical disturbances of a magnitude which can be detected by our apparatus are set up. It is necessary that the receiver and the transmitter should be in tune. Lodge made use of this principle, and, by receiving the waves on wires adjusted to resonance with his Leyden jar and coil, was able to detect them. David Hughes, working in the early eighties, had already detected such oscillations, but was discouraged from pursuing the subject.

In 1879, in consequence of the offer of a prize by the Berlin Academy, the attention of Heinrich Herz, then a student under Helmholtz, was drawn to the problem of electric oscillations and their detection. He came to the conclusion that with the means of observation then at his disposal "any decided effect could scarcely be hoped for, but only an action lying just within the limits of observation." The investigation was laid aside only to be revived in 1886 by a chance observation of the effect of resonance in two circuits which happened to be in tune, and his realization of the fact that herein lay the means of solution of his problem. The great man and the not-too-small theory of the first James Forrest Lecture had met.

His Paper "On Very Rapid Electric Oscillations" appeared in Wiedemann's "Annalen," vol. xxxi, for 1887, and from the experiment came verification of Maxwell's theory, the basis of all our knowledge of wireless.

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[1] "*Über die Erhaltung der Kraft,*" 1847.

Fitzgerald called the attention of English physicists to the work at the British Association Meeting in 1888, and Lodge exhibited many of the effects of the waves at the Royal Institution in 1889. The investigations which led to such brilliant results were inspired by the desire for knowledge; the idea of their practical application was entirely absent; signalling by wireless waves was not foreshadowed until Crookes suggested it in 1892, and in 1893, the year of Sir William Anderson's Lecture, Lodge heard of Branly's coherer and applied it to the rectification and reception of wireless waves. From this started the investigations of many of those whose names as pioneers are familiar to all. But another discovery in pure science was necessary to complete the work.

Edison had shown in 1883 that if an insulated electrode was inserted in an ordinary glow-lamp there was a current of negative electricity from the filament to the electrode—the emission of negative electricity from a hot body had been observed by various experimenters—and Fleming made some observations about this date on the Edison effect. In 1904 he applied them to produce a valve rectifier for high-frequency oscillations by connecting one pole of his receiving circuit to an insulated plate or cylinder within a carbon lamp, of which the negative electrode forms the other pole of the receiving circuit. When the filament is made incandescent, negative electricity can readily pass from it to the insulated plate and hence into the receiving circuit; the flow of positive electricity in the same direction is checked; the lamp has a rectifying action.

Dr. Lee de Forest improved this oscillation valve a little later, making it an amplifier as well as a rectifier by placing between the filament and the plate or cylinder a grid of metal wire connected to an external source of electromotive force, by means of which its potential can be varied. There is ordinarily a current of negative electricity passing from the filament to the plate—the plate current it is called—through the interstices of the grid. By varying the potential of the grid this current can be varied, and the conditions can be so adjusted that small changes in the potential of the grid will produce large changes in the plate current; the plate current is passed through the primary of a step-up transformer, in the secondary of which is the receiving telephone, and the effect is thereby made audible. The grid is connected to one pole of the circuit receiving the incoming waves, and the small variations of potential which they produce thus give rise to large variations of the plate current, and hence the sound is amplified. By placing a number of valves in series very large amplifications are possible.

The other uses of the valve are very numerous. It is now employed as a transmitter for wireless work; while it finds many applications as a source, or rather regulator, of vibrations of comparatively short period. The Post Office has used it as an amplifier of speech, while Mr. F. E. Smith has applied it as a source of sound in connexion with the measurement of audibility. The whole of this arose from Edison's observation of the discharge of negative electricity from the heated filament.

To quote again from the first Forrest Lecture: "The Engineer must banish from his mind the idea that anything can be too small or too trifling to deserve his attention." For the modern development of the valve, through the researches of those who have brought it to its present excellence, has rested on a still smaller entity, the electron, a body with a mass of $0{\cdot}900 \times 10^{-27}$ grams, about 1/1800 of the atom of hydrogen, carrying a negative charge of $1{,}591 \times 10^{-20}$ electro-magnetic units[1] of electricity, first glimpsed by Crookes, then proved to exist by J. J. Thomson.

The appearance of a Crookes tube or vacuum tube when carrying an electric discharge is well known. When the pressure is sufficiently reduced, the tube is non-luminous, except for a beam of light which proceeds normally from the cathode—the negative electrode—and penetrates into the tube a distance depending on the pressure; this beam constitutes the cathode rays; if the rays strike the glass at the end of the tube, a vivid fluorescence is produced.

Crookes showed that the beam constituted a current of negative electricity; it could be deflected by a magnet. Experiments by Perrin and J. J. Thomson proved conclusively the existence of the negative charge. Thomson showed also that the stream consisted of an assemblage of minute particles—electrons. He measured the velocity of the particles and the ratio ϵ/m of the charge on each to its mass. Further experiments, of which perhaps those of Millikan are the most important, have led to a determination of the charge on the electron, and from this and a knowledge of the ratio the values of ϵ and m, given above, are found. And these values are the same whatever be the nature of the cathode from which the rays take their origin—the mass and charge of an electron are the same whatever be its source. Thus now it is hardly too much to say that nearly all electrical phenomena are conditioned by the presence and motion of electrons. The current in a cable is a stream of electrons; a conductor is a body through which they move freely; an insulator checks their activity. The power that drives our motors comes from them; the light of the electric lamp, the heat that comes from an electric radiator, have their origin in these tiny particles; the plate current of the valve rectifier referred to above is a stream of electrons; when the grid is negatively electrified, it adds negative electrons to the stream; when it is positive, some of the electrons from the filament are stopped, in their passage through its interstices, to neutralize the positive electricity it possesses.

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[1] One electro-magnetic unit is the charge transferred by 1 ampere circulating for 10 seconds.

Electrical engineering in its many branches is closely bound up with the properties of an electron discovered by men whose sole object it was to advance natural knowledge. Nor is this all: for from the electron came X-rays, though this, perhaps, is hardly the correct way of putting it, as J. J. Thomson's discovery really followed that of Röntgen. About 1894 physicists in many countries were experimenting with Crookes cathode rays. A chance observation made by a skilled worker revealed the fact that the cathode rays produced an effect outside the tube in which they were generated. Röntgen in the autumn of 1895 was conducting an investigation with a vacuum tube wrapped in light-proof paper and noted that a fluorescent screen of barium platino-cyanide lying near shone out when the tube was excited; if he placed opaque objects between the screen and the tube, shadows were cast on the screen, showing that rays, the X-rays, proceeded from the tube in straight lines; while it was quickly found that the rays penetrated substances opaque to light, the penetration depending on the density of the substance. There is no need to dwell on the results that have followed from this, and their significance to engineers. X-rays can penetrate 4 to 5 mm. of lead, 12 mm. of tin, 75 mm. of carbon steel, 100 to 150 mm. of aluminium, and 300 to 400 mm. of wood. By their aid hidden cracks or faulty welds can be shown upon metal structures, while they have been employed for many purposes, besides their use in surgery and medicine.

For some time their nature was a mystery. Their rectilinear propagation and the absence of refraction when they fell obliquely on the surface of a medium other than air were difficult of explanation. Now it is known that they are produced by a very rapid change of motion of electrons. When the velocity of an electron is altered, an electro-magnetic wave is produced, and, starting from the electron, travels outward with the velocity of light. The frequency in this wave—in the number of vibrations per second produced—depends on the rate of change of velocity of the electron. If this is high, the frequency in the resulting wave is very great. When a beam of cathode rays falls on the glass walls or on the anti-cathode of an X-ray bulb, the electrons are stopped almost instantaneously. Electro-magnetic rays of very high frequency—X-rays—are produced. Their wave-lengths are now known to lie[1] between 12×10^{-8} cm. and $0{\cdot}17 \times 10^{-8}$ cm. The wave-length of visible light is between $7{,}700 \times 10^{-8}$ cm. and $3{,}600 \times 10^{-8}$ cm., that of ultra-violet light lies between $3{,}600 \times 10^{-8}$ cm. and 200×10^{-8} cm., and it is to this minuteness of wave-length that the absence of refraction is due. In the hands of Sir William and Professor W. L. Bragg, it has been the means of revealing the inner structure of materials in a manner of the utmost importance to engineers. We will return to this later.

Turning now to a less abstract branch of electrical science, a research of great importance to all users of electricity has just been carried through by Mr. S. W. Melsom and his assistants at the National Physical Laboratory with the co-operation of Mr. E. Fawssett at Newcastle. Its object was to determine the permissible current-loading of British Standard cables laid in various ways in the ground, and it was undertaken as part of the work of the British Electrical and Allied Manufacturers' Association already referred to.

The problem is to determine the maximum current which the cable can carry without raising the temperature above some specified figure, taken as 65° C. for armoured cables laid direct in the ground, and 50° C. for plain lead-sheathed cables drawn into ducts. In forming the tables with which the research concludes, it is assumed that the average temperature of the ground is 15° C.

The heat generated in the cable depends on the current and resistance, while the temperature-rise depends on the heat generated, the dimensions and specific heat of the copper core, the thermal resistance of the insulating material between the core and the outer covering, and the thermal resistance of the ground surrounding the cable. The measurement of these quantities is in no case easy; the calculation of the thermal resistance of the insulator when once its resistivity has been determined is simple for a single-core cable, but complex in the case of three cores; the resistivity of the ground depends on the material of the soil, the amount of moisture present, and the depth of laying; but in the end tables have been drawn up which in the opinion of Mr. Llewellyn Atkinson, the late President of the Institution of Electrical Engineers, will save users tens of thousands of pounds annually, besides reducing the initial cost of the equipment. *Figs.*[1] *3* and *4* show some of the results attained for continuous and intermittent loading respectively.

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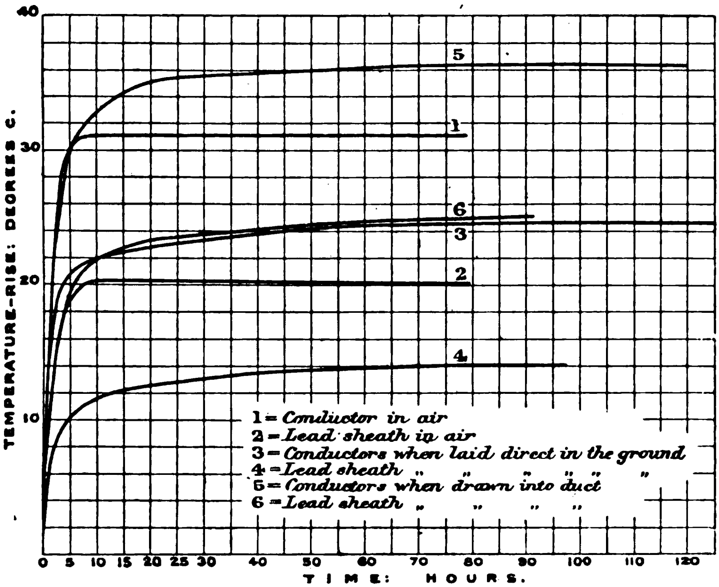
[1] See "Dictionary of Applied Physics," vol. iv. Radiology.

Few matters are of more importance to the engineer than the properties, metallurgical and other, of the materials he uses in his structures, and for the knowledge of these he owes everything ultimately to the researches of the seeker after new knowledge. From the time when Sorby applied the microscope to the study of metals and alloys, and Roberts-Austen and Osmond investigated their heating and cooling curves, to the present day, there has been continuous progress. Much of this is due to the investigations of the Alloys Research Committee of the Institution of Mechanical Engineers, and a comparison of the earlier reports of that Committee with the eleventh report just issued from the National Physical Laboratory shows in an interesting manner the extent to which modern industrial processes rest on pure science and how great is the debt incurred since those early days.

Fig. 3.



For some time the Committee has worked on light alloys, impelled to it in part by the need to develop these for use in aircraft. It is good to learn that, thanks in great measure to the generosity and zeal of Sir John Dewrance, the Committee has been reconstituted so as to be more widely representative, and intends in the future to direct its work to the study of ferrous alloys. It is hoped to prepare alloys of iron with other metals, such as chromium and cobalt, which shall be free from carbon, and then to go on to more complex materials. As the memorandum, in which Dr. Rosenhain brought before the Committee the proposals for this work, states:— "It may confidently be expected that work of this kind, which has never been attempted in this systematic manner before, is likely to lead to results of a far-reaching importance and to the evolution

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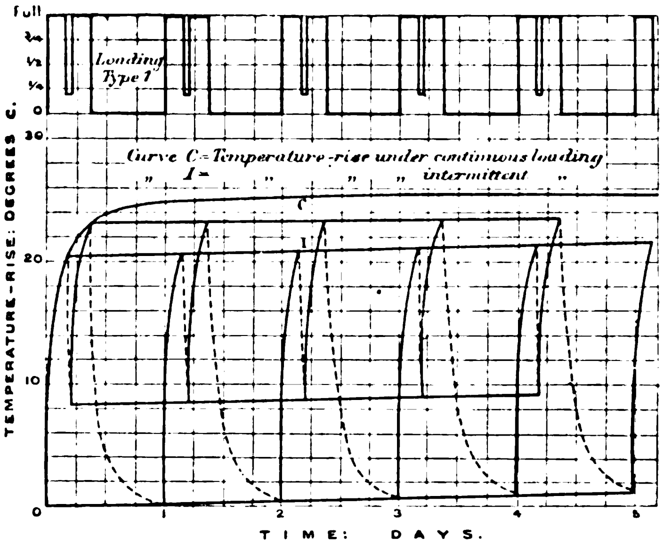
[1] Reproduced by permission from the Second Report to the Institution of Electrical Engineers.

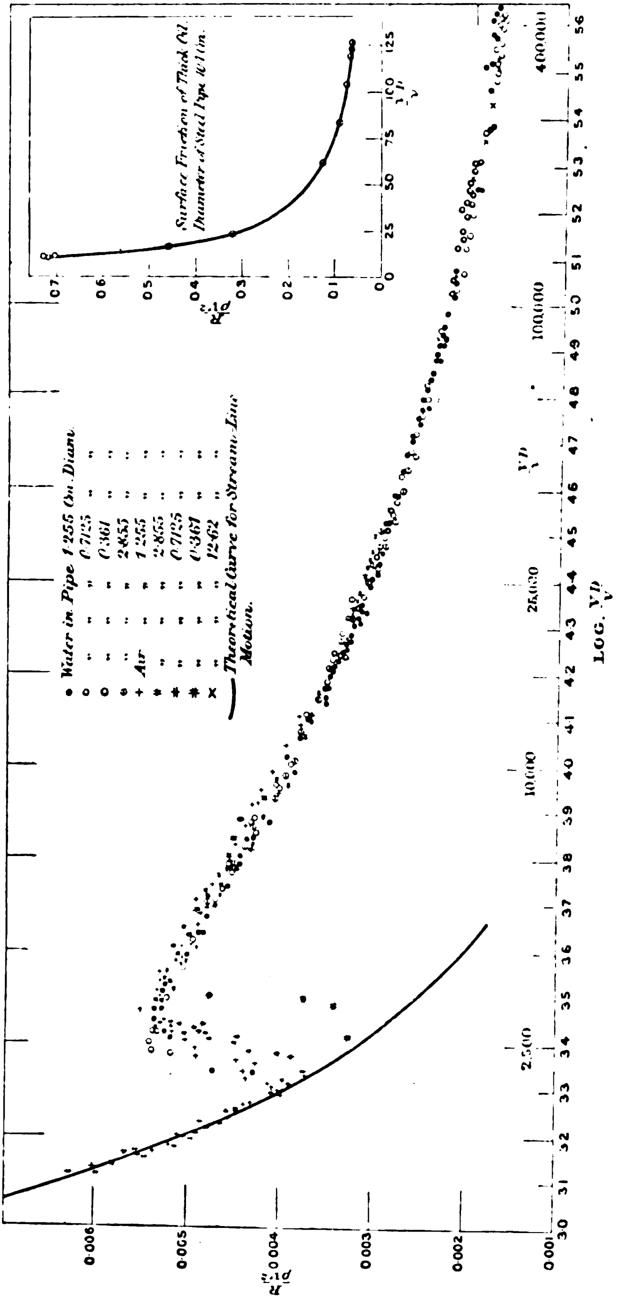
Fig. 4.



of materials likely to be of very special value to the mechanical engineer." To me it is a matter of very real pleasure that the greater part of the work will go on at the National Physical Laboratory.

The records of the engineering department of the Laboratory are full of investigations which illustrate the interdependence of abstract science and engineering. Dr. Stanton has been kind enough to help me to obtain details of some few. The resistance of a body moving through a fluid in which it is completely immersed is either proportional to the relative speed of the two or to a power of the speed very approximately equal to 2; and the

Fig. 5.

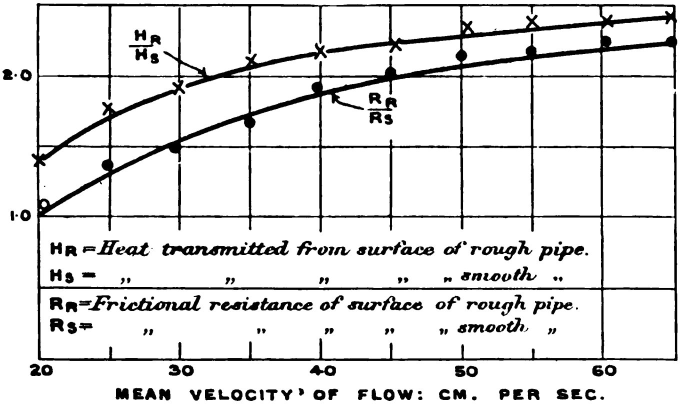

● Water in Pipe 1·255 in. Diam.
○ ,, ,, ,, 0·7125 ,,
O ,, ,, ,, 0·361 ,,
⊕ ,, ,, ,, 2·855 ,,
+ Air ,, ,, 1·255 ,,
✱ ,, ,, ,, 2·855 ,,
✱ ,, ,, ,, 0·7125 ,,
✱ ,, ,, ,, 0·361 ,,
× ,, ,, ,, 12·62 ,,
Theoretical Curve for Stream-Line Motion.
Surface Friction of Thick Oil.
Diameter of Steel Pipe 1·011 in.
0·7
0·6
0·5
0·4
0·3
0·2
0·1
0
25
50
75
100
125
$\frac{VD}{\nu}$
$\frac{R}{\rho V^2}$
0·006
0·005
0·004
0·003
0·002
0·001
3·0
3·1
3·2
3·3
3·4
3·5
3·6
3·7
3·8
3·9
4·0
4·1
4·2
4·3
4·4
4·5
4·6
4·7
4·8
4·9
5·0
5·1
5·2
5·3
5·4
5·5
5·6
2,500
10,000
25,000
100,000
400,000
$\frac{VD}{\nu}$
LOG. $\frac{VD}{\nu}$

line dividing the two is clearly marked. When the motion is stream-line in its nature we have to deal with the direct power of the speed; when, however, it is turbulent or eddying, the square of the relative speed comes in. Osborne Reynolds showed, as is well known, by experiments on the flow of liquids down a long pipe. that the change from previous steady motion to turbulent occurred when the quantity vd/ν exceeded a certain value which is about 300; in this expression v is the relative speed, d the diameter of the pipe, and ν the kinematic coefficient of viscosity. Turbulent motion, it is found, resolves itself into steady when vd/ν exceeds 2.000, and this value gives Reynolds's critical velocity. Any physical quantity can be expressed in terms of the fundamental units of mass, length, and time. Each of these units may occur in its expression raised to some definite power. If this is done, all the terms in any equation having a physical significance must necessarily have identical dimensions in each of these units respectively. If one term be of the form $a\,M^l\,L^m\,T^n$, where a is a numerical constant, M, L, and T represent the units of mass, length, and time, and l m, n are constants, then any other term in the expression must be of the form $b\,M^l\,L^m\,T^n$, where b is some other numerical constant.

Lord Rayleigh, by an application of this principle, showed that the resistance to the motion of a body through a fluid, assuming no forces to act on the fluid, must be of the form $kV^2D^2f\left(\frac{VD}{\nu}\right)$ where k is a numerical constant, V the relative velocity, D a length depending on the size of the body, and ν the kinematic viscosity, which is equal to μ/ρ, where μ is the viscosity as ordinarily defined, and ρ the density of the fluid. This result of abstract theory is the basis of all model work in aeronautics, and in this connection will be referred to later. It was applied by Stanton to harmonize the results reached by himself and many investigators who had measured the resistance to flow of fluids in pipes. The friction per unit area, denoted by R, will be of the form $k\rho V^2f\left(\frac{VD}{\nu}\right)$, and in the curve shown in *Fig. 5* the quantity $R/\rho V^2$ is plotted against log VD/ν for values of air and water above the critical speed of Reynolds's formula, extending from $VD/\nu = 2{,}500$ to $VD/\nu = 470{,}000$. The curve is a complete verification of Lord Rayleigh's formula; the resistance is clearly a function of VD/ν; below the critical value, the left-hand branch is in good accordance with the hydrodynamical equation $\mathrm{R} = \frac{8\mu V}{D}$; the right-hand branch shows a value for $R/\rho V^2$ gradually decreasing as VD

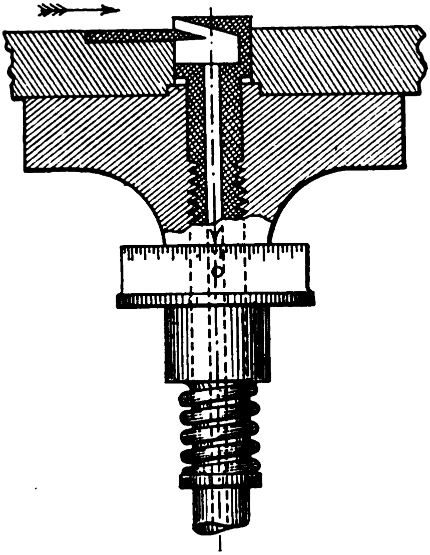
increases and tending to become a straight line parallel to the axis of abscissæ; the resistance tends to become proportional to the square of the speed.

Fig. 6.



Again, it follows from a further application of Reynolds's theory that the rate of transmission of heat from a heated pipe to a fluid

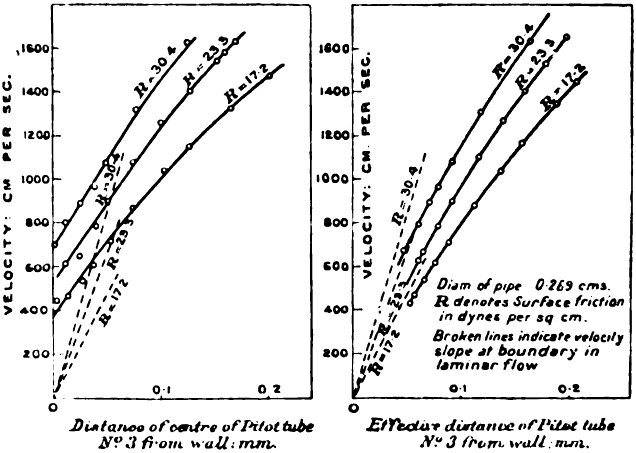
Fig. 7.



flowing through it, or vice versa, depends on the roughness of the pipe. *Fig. 6* illustrates this, and shows that the two curves, the one giving the ratio of heat transmitted, the other the ratio of the frictions in two pipes of the same size and material, of which the surface of one had been artificially roughened, are closely parallel. Once more theory shows that, even when the fluid in a pipe is in turbulent motion, the layer in contact with the surface is at rest, and there is a thin layer adjacent to this in which the motion is laminar. *Figs. 7* and *8* illustrate experiments by which this fact was verified, the one (*Fig. 7*) showing a special Pitot tube arranged to measure the pressure, and hence the velocity on the assumption, of stream-line motion up to within 0·1 mm. from the tube wall, the other (*Figs. 8*) the result of these measurements.

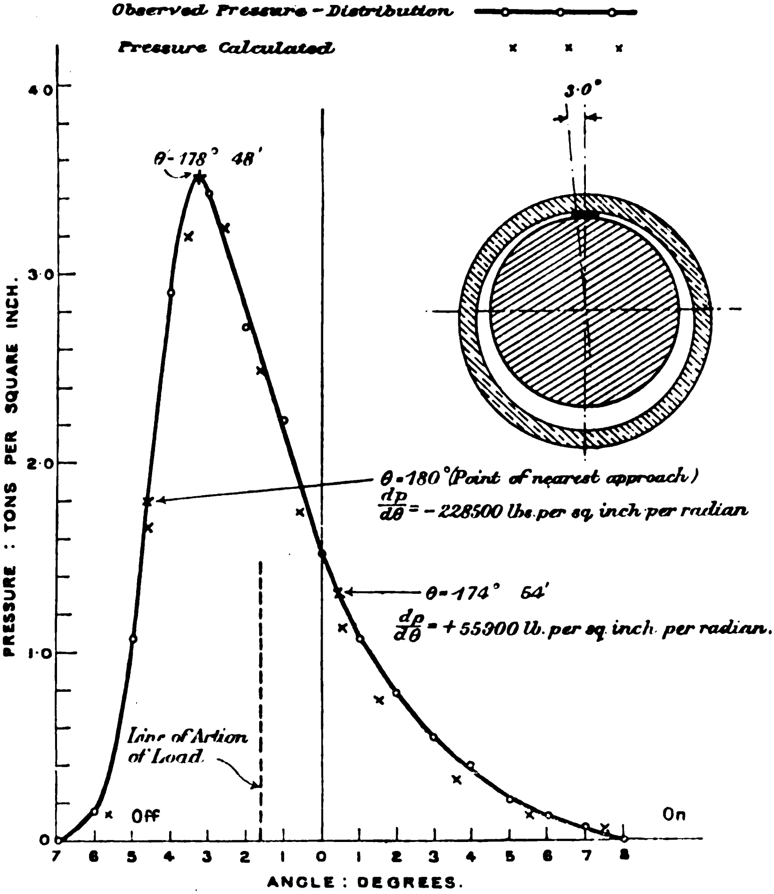
Closely connected with these results, and even of more importance perhaps to engineers, is the investigation on lubrication described by Dr. Stanton in his recent Thomas Hawksley Lecture before the Institution of Mechanical Engineers. And here, again, we have recourse to the theoretical work of Osborne Reynolds, undertaken with a view to explaining the results obtained by Mr. Beauchamp Tower in his experiments between 1883 and 1885. Reynolds studied the conditions existing between a revolving journal and a half bearing, under which the known laws of hydrodynamics would permit a film of fluid to exist between the two, such that the pressure-distribution in the film would account for the load and the shear account for the friction; and he obtained an equation connecting the pressure-changes in the film and the other quantities involved. Harrison has since extended the mathematics to the case of a

Figs. 8.



complete bearing. The problem requires a knowledge of the difference of radii of journal and bearing, and of the temperature and viscosity of

Fig. 9.



Diameter of Bearing	*1·06 inch.*
Length of Journal	*2·50 inches.*
Diameter of Journal	*1·00 inch.*
Temperature of Film	*49·4° C.*
Speed	*1,000 R.P.M.*
Total Load	*690 lbs.*
Arc of contact of Oil Film	*15 degrees.*

Least distance between surfaces of Journal and Bearing = 0·000046 inch at 3·0 degrees to left of Vertical through centre of Journal.

the oil-film. Mr. Hyde's recent experiments at the National Physical Laboratory give the relation between the viscosity and the pressure.

Dr. Stanton has recently succeeded in measuring the pressure in the film over an arc of contact of from 14 to 15 degrees, for various oils, under a total load of 690 lbs., at a journal speed of 1,000 revolutions per minute. The maximum pressure for castor oil was 3·5 tons per square inch, and the resultant of the vertical components of the pressure approximately balanced the load. At their nearest point of approach the surfaces were only 0·000046 inch apart, while the coefficient of viscosity agreed with that found by Mr. Hyde. Thus the experiments showed that the laws of hydrodynamics hold, and are competent to account for the observed facts, even in a film which is $\frac{5}{100000}$ inch in thickness, and which is subject to a pressure of 3·5 tons per square inch. *Fig. 9*, taken by permission from Dr. Stanton's Paper,[1] gives the curve of pressure for castor oil.

Of great interest to any engineer is a knowledge of the elastic properties of the material with which he deals, and of the changes which go on in the material when the limits of its elasticity are exceeded. For the study of these properties various forms of extensometer have been devised. Professor Dalby's work in this field is well known, and he has himself described his load–extension elastic instrument, which, by a skilful combination of the principle of the weighbar and magnification by the aid of photography, gives a complete record of the elastic properties of the specimen under test in the course of a few minutes and in a form which can be readily reproduced and kept for study at leisure.

One such diagram[2] is shown in *Fig. 10*. Starting from O with a tension, the spot moves to A along the straight line OA; on reversing and putting on a compressive load it moves to B, and so long as the elastic range is not exceeded, it traverses the line AB accurately. But if the tension stress is increased beyond A the spot moves up to C and then rapidly falls to D; the specimen is broken down. On reducing the load at D to nothing, the spot moves to E, DE being parallel to the elastic line AB, and on putting on a compression the spot moves to F. On removing the compression, there is motion to G parallel again to the elastic line, and on applying tension we have motion from G to H. By repeating the operations we get a second similar loop HIJ, but of larger area. The breaking down of the material is accompanied by the formation of slip bands described long since by Ewing and Rosenhain, and Professor Dalby has described in a diagrammatic way changes due possibly to the sliding of the material along the slip bands and its elastic recovery, which give generally an explanation of the results shown in the figure.

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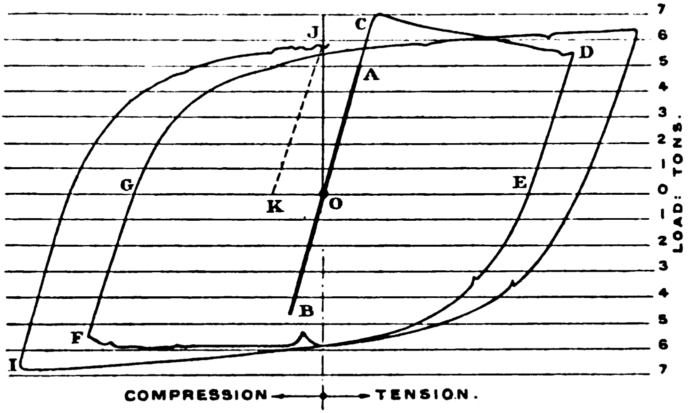
[1] Proc. Inst. Mech. E., 1922, vol. ii, p. 1138.

[2] Reproduced by permission from Proc. Royal Soc., Series A, vol. ciii (1923), p. 13.

But another Paper, read recently as the Bakerian Lecture to the Royal Society by Mr. G. I. Taylor and Miss Elam, enables us to see further into the mechanics of strain. The action of a diffraction-grating is well known; the grating consists of a plate of polished glass or metal on which is ruled a series of fine lines at equal distances apart at a rate of from say 6,000 to 15,000 to the inch. Thus the distance apart of the lines of a grating may be of the order 15,000 × 10^{-8} cm., or say from two to four times the wave-length of visible light.

When a beam of homogeneous light falls normally on such a grating from a small source, preferably a narrow slit parallel to the lines of the grating, a series of bright images of the source is formed. If A be the angle which the line forming one of these images to the centre of the grating makes with the normal, λ the wave-length of light and ϵ the distance between the grating lines, then theory and experiment both show that the relation sin $A = m\lambda/\epsilon$ holds where m has the values 1, 2, 3, etc. These images are said to be diffracted images of the source, and it is clear that if they are to be formed, λ and ϵ must be quantities of the same order, while the equation can be used to determine λ if ϵ is known, or vice versa. Similarly any natural material reflecting light from a surface regularly marked with a fine structure resembling the lines of a grating, or arranged as a series of thin films of varying reflective power,

Fig. 10.



can produce diffraction effects, and the colours of various material objects are due to these.

X-rays, however, are not diffracted by any artificial grating, however fine, and we see the reason. The wave-lengths of the rays lie between 12×10^{-8} cm. and $0{\cdot}17 \times 10^{-8}$, being only from one to ten thousandths of the distance between the lines of the grating; thus the diffraction angles are far too small to be observed.

But a crystal owes its regular form to the regular spacing of the atoms of which it is composed, and the distance between the various planes of symmetry in which these atoms lie is of the same order as the wave-lengths of the X-rays. It occurred to Laue in 1912 that this regular spacing might provide a natural medium suitable for the diffraction of X-rays, and he predicted the directions in which diffraction would take place, making certain assumptions as to the nature of the spacing of the crystal atoms. This was verified in 1913 by Frederick and Knipping.

The theory was carried a stage further by W. L. Bragg in a Paper in the Cambridge Philosophical Proceedings for 1912, and in the hands of his father, Sir W. H. Bragg, and himself has developed into a method by which the molecular configuration of the crystal can be determined, and changes produced by alterations in its form can be measured.

It is not possible, however, to apply this to a piece of steel under stress. The steel consists of a multitude of microscopic crystals embedded in a non-crystalline cement; their axes and planes of symmetry are oriented in all directions, and regular diffraction in any one direction can never be observed. Much work, however, has been recently done on aluminium and its alloys, and Dr. Carpenter and Miss Elam, working at the Imperial College, discovered a method of producing large specimens of aluminium containing only a small number of crystals; in their final experiments a number of bars of diameters 0·564 and 0·798 inch were so treated that each consisted of one single crystal. Tensile tests on bars consisting of but a few crystals showed remarkable results. The problem of determining the state of strain in such a crystal when under stress was then attacked by Mr. G. I. Taylor, and in the Bakerian Lecture he and Miss Elam describe the results of gradually straining a test-piece of rectangular section and dimensions $1{\cdot}0 \times 1{\cdot}0 \times 20{\cdot}0$ cm. The faces were marked with lines parallel to their length and cross lines oblique to these, and gradually strained in eight stages up to an extension of 72 per cent. At each stage the angle between the faces, the distances between the cross lines, and the angles these lines make with the longitudinal axis were measured, while the positions of the crystal axes were also determined by aid of X-rays. Thus the distortion of the crystal was determined and also the position of the crystallographic axes relative to the direction of the stress. The conclusion was reached that up to an extension of 40 per cent. the distortion took place entirely by slipping along a plane equally inclined to the three principal axes of the crystal, a plane denoted by the symbol (111) as employed by crystallographers.

Now there are four such planes parallel to the faces of a regular octahedron, and on each of these there are three crystallographically-similar possible directions of shear. It was found that the slipping took place along that possible direction of shear in which the shearing force tending to produce motion is of maximum value. In the end the result of the successive slips was that the crystal axes so placed themselves that two of these octahedral planes then made equal angles with the axis of the specimen. The figures show that this had probably happened before the last strain which brought the total extension up to 72 per cent., and that then slipping occurred along both these planes, with the result that the specimen broke with a chisel edge instead of the usual conical fracture of an ordinary test-piece. Thus we may perhaps conclude that in each of the crystals composing a steel test-piece there are certain planes and directions along which slipping can take place. As the test-piece is strained, each crystal slips along that plane and in that direction in which the component of the force producing the strain is greatest. The final strain is the resultant of all these microscopic slips. So long as the strain is not too great, the crystal is elastic and recovers its original form when the stress is removed. X-rays have in this way revealed to us much that was hidden in the mechanism of stress and strain.

Thirty years ago Sir William Anderson referred to the phenomenon of fatigue in engineering materials, as shown in the breaking of locomotive-axles, screw propeller-shafts, and other parts subject to alternating or rapidly-varying conditions of stress. Since that time much abstract science has been applied in the attempt to elucidate the problem, and many methods of determining the fatigue limits have been devised. It must suffice here to refer to the ingenious model[1] by the aid of which Professor Jenkin of Oxford has been able to illustrate the mechanism of failure and give a reasonable explanation of many of the principal phenomena observed, and to mention the modification of the well-known Wöhler method of determining the fatigue limits, devised by Mr. Gough of the National Physical Laboratory and illustrated in *Fig. 11.* A stainless-steel mirror is attached to the specimen in such a manner that it is at right angles to the axis. A small source of light which can move along a vertical scale is adjusted so that its image is visible in a telescope having its axis horizontal and directed to the mirror.

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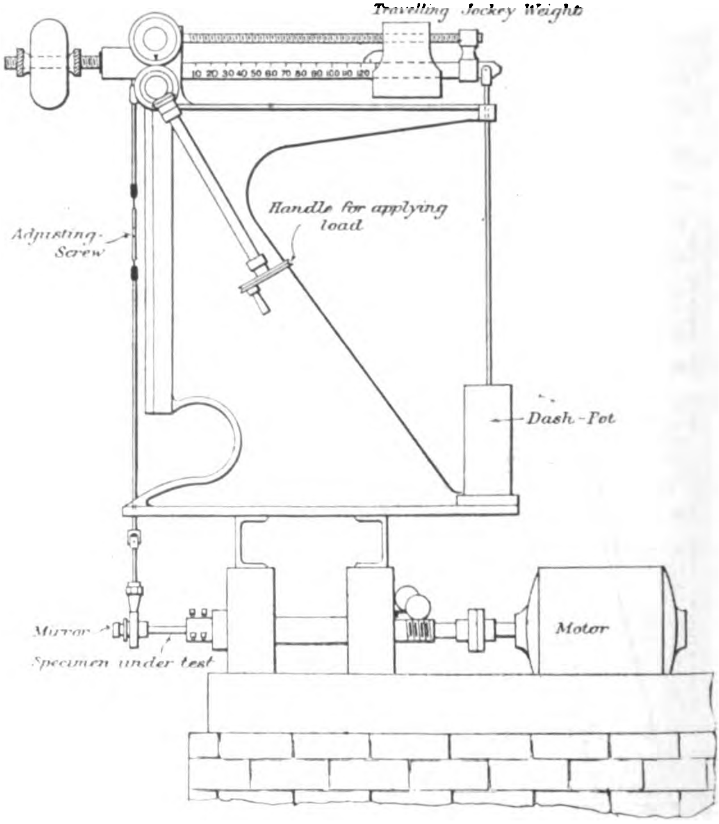
[1] Proceedings Royal Soc, vol. ciii (1923), p. 121.

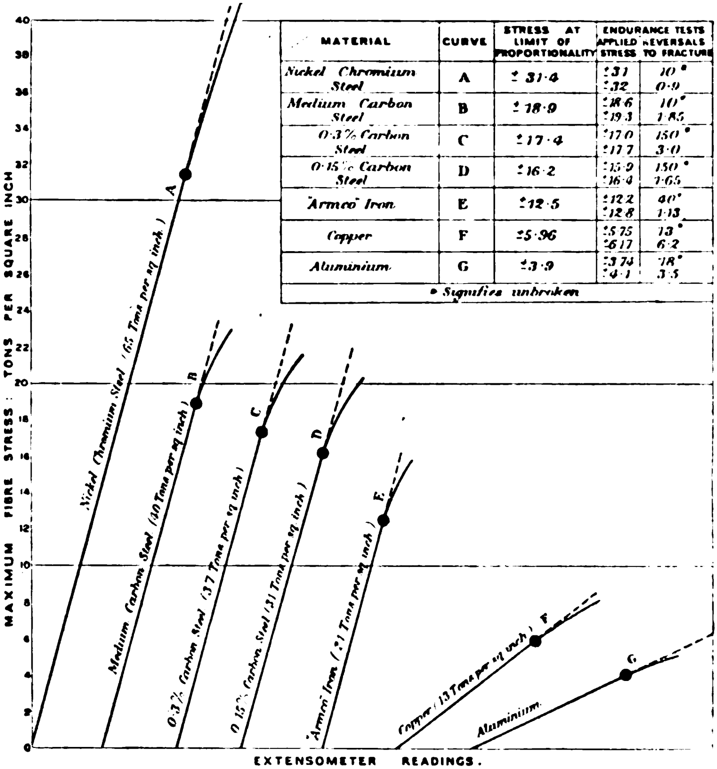
Fig. 11.


Travelling Jockey Weight
10 20 30 40 50 60 70 80 90 100 110 120
Handle for applying load
Adjusting Screw
Dash-Pot
Mirror
Specimen under test
Motor

When the specimen is unloaded, the light remains visible as it rotates. If a load be gradually applied, the specimen bends, and the light is shifted so as always to remain on the cross wire of the telescope. Its position can be read to 0·01 inch. If the scale-reading is plotted against the load, a straight line is at first obtained followed by a deviation turn when the limiting range of stress is reached. The time taken for an observation varies from 10 minutes to half an hour, depending on the rate of loading; but in any case the reduction of time from that needed for the old method is very great. *Fig. 12* shows some of the curves obtained.

No branch of engineering owes more to abstract science than aeronautics. Lord Rayleigh's work on dynamical similarity is at the base of all wind-channel work on models. In a wind-channel the forces on a model aircraft due to its motion relative to the air, or the thrust and torque of a model propeller, are measured, and from these, using the principle of dynamical similarity, the corresponding full-scale effects are inferred from the assumption that they are proportional to the square of the linear dimensions and the square of the speed. Strictly this implies, as we have seen, that the quantity $f\left(\frac{VL}{\nu}\right)$ has the same value for model and full-scale. Thus either this

Fig. 12.



MATERIAL	CURVE	STRESS AT LIMIT OF PROPORTIONALITY	ENDURANCE TESTS APPLIED STRESS	REVERSALS TO FRACTURE
Nickel Chromium Steel	A	± 31·4	±31 ±32	10 • 0·9
Medium Carbon Steel	B	± 18·9	±18·6 ±19·3	10 • 1·85
0·3% Carbon Steel	C	± 17·4	±17·0 ±17·7	150 • 3·0
0·15% Carbon Steel	D	± 16·2	±15·9 ±16·4	150 • 1·65
"Armco" Iron	E	± 12·5	±12·2 ±12·8	40 • 1·13
Copper	F	± 5·96	±5·75 ±6·17	13 • 6·2
Aluminium	G	± 3·9	±3·74 ±4·1	18 • 3·5

• *Signifies unbroken*

quantity is constant or $\frac{VL}{\nu} = \frac{V_1 L_1}{\nu_1}$, where the subscript refers to the full scale. Since air is in both cases the surrounding medium $\nu = \nu_1$ and we must have $VL = V_1 L_1$. This, however, is impossible if L is necessarily much less than L_1, and it is impossible to make the speed in the wind channel proportionately greater than that of the full-scale machine. We are driven, then, to the first alternative, that for a strict comparison $f\left(\frac{VL}{\nu}\right)$ must be a constant.

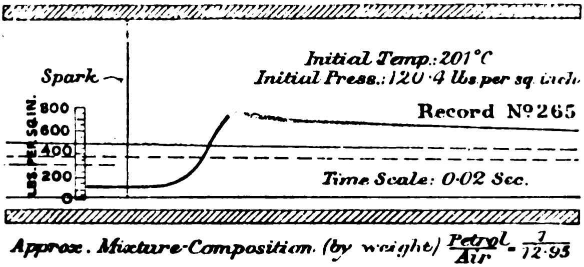
Experiments such as Dr. Stanton's on the flow of air and water in pipes, and the actual measurement of the forces on models at different wind-speeds, show that this is very approximately true in most cases. Airship models of good performance form an exception to this statement; but the corrections required to model results are now generally accepted, and there is a gratifying agreement both between different wind-channels and the conclusions from model and full-scale experiments. However this may be, the problem of the flow of air round an aeroplane is no easy one, and there is ample opportunity for abstract science to influence still more engineering practice.

The development of the internal-combustion engine is another engineering problem which owes much to exact science. Its foundation is contained in the celebrated memoir of Sadi Carnot: "*Réflexions sur la puissance motrice du feu et sur les machines propres a développer cette puissance*," published almost exactly 100 years ago—Paris, 1824—in which he sets out "to establish reasoning applicable not only to steam engines, but to every imaginable heat engine, whatever be the working substance and whatever be the manner in which it is employed."; and is led to the remark, "wherever there exists a difference of temperature there can be a production of motive power."

It is a long journey from this, either through the steam engines of his day—Carnot refers in particular to the high-pressure engines of Trevithick and Vivian (*deux habiles ingénieurs anglais*)—working at pressures of 8 or 10 atmospheres, but without condensers—to the turbine which now drives our 20,000-ton liners, or through the primitive hot-air engine of Stirling or the early Otto machines, to the Rolls-Royce and the Napier "Lion" of modern aeronautics. But abstract science has marked every step of the journey, and progress has been rendered possible only by the labours of many investigators who have determined such necessary qualities as the heat of combustion of the materials used as the working substances and their specific heats up to temperatures of 2,000° C., and have measured the rate of propagation of flame through the gases produced, its variation with the pressure, and the effect of small changes in the exploding mixtures. Superposed on these have been inquiries into the distribution of temperature in the cylinder, the rate of loss of heat by conduction and radiation, and many metallurgical investigations with a view of finding material suitable to stand the heavy stresses imposed on pistons and other moving parts and yet sufficiently light to satisfy the demands of the designer of the aircraft.

In this work many have combined to help; perhaps I may mention specially the Light Alloys Sub-Committee of the Aeronautical Research Committee, the work of the Metallurgical Department of the National Physical Laboratory and of Professor Lea of Birmingham, with the Internal-Combustion Engine Committee of the British Association, which numbers among its members men so well known to this Institution as Sir Dugald Clerk, Chairman, Professors Callendar, Coker, and Dalby, and in days gone by inspired much of the work of Bertram Hopkinson on the explosion of gases. To Callendar we owe the platinum thermometer, which has added so greatly to the accuracy of thermometric work. His work and that of Dalby, with his ingenious device for inserting the thermometer in the cylinder at the right instant, have rendered possible the determination of the temperature at each stage of the engine-cycle, provided we can draw the indicator-diagram, and this the researches of Dalby again have made possible. As a result of all this we can obtain in any case the materials for drawing up a heat balance-sheet for the engine; while in quite recent years the work of Ricardo has indicated means for securing a greater efficiency, especially at high altitudes, and much light has been thrown on the problem of detonation by his investigations, combined with those of Fenning at the National Physical Laboratory. *Fig. 13*

Fig. 13.



shows one of the explosion curves obtained by the latter during the explosion of an air-petrol mixture in a closed cylindrical vessel, 7 inches in diameter by about 8 inches in length. The mixture, prepared in a separate vessel, was ignited in a stagnant state by a high-tension spark situated approximately at the centre of the explosion cylinder, a secondary spark gap being employed to record the passage of the spark on the diagram.

The record clearly illustrates (1) the delay between the passage of the spark and the apparent commencement of any rise of pressure, (2) the comparatively slow rise of pressure, and (3) a sudden rise just before the maximum, resulting in a metallic ring and corresponding, no doubt, to "pinking" in an actual engine. The latter portion of the cooling curve is a broken line, due to the operation of an interceptor actuated by a tuning-fork to obtain the time scale, the distance between corresponding points on alternate dashes representing about $\frac{1}{50}$ second.[1]

And, to return to the steam engine, Sir William Anderson, in his lecture, referred to the rude and cheap apparatus with which Watt, in the astonishingly short period of 2 years, "furnished himself with the abstract knowledge required for explaining in a definite manner the action of the steam engine," and then "had no difficulty, as soon as his theoretical ground was sure, in determining what mechanical arrangements were necessary to realize the conditions imposed by science."

I have neither the time nor the skill to trace in detail the steps by which the genius of Parsons was led to develop the turbine, or to narrate the history of its growth from the "Turbinia" to the present day.

The main facts are these :—In 1897 the "Turbinia," of 44½ tons

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[1] Further particulars are :—

Fuel : Shell Aviation Spirit.
Initial pressure of charge : 120·4 lbs. per square inch (absolute).
Maximum pressure after ignition : 764 lbs. per square inch (absolute).

$$\frac{\text{Maximum pressure}}{\text{Initial pressure}} = 6\cdot35.$$

Pressure $\frac{1}{10}$ second after maximum : 636 lbs. per square inch (absolute).
Initial temperature of charge : 201° C.
Explosion time (i.e., passage of spark to maximum pressure) : 0·049 second.
Time interval from apparent commencement of rise to maximum pressure : 0·0404 second.
Ratio of air to petrol by weight = 12·95 to 1.

Partial analysis of cooled products of combustion $\begin{cases} CO_2\text{—}12\cdot6 \text{ per cent.} \\ O_2\text{—}0 \text{ per cent.} \\ CO\text{—}3\cdot65 \text{ per cent.} \end{cases}$

displacement, 100 feet in length, attained a speed of 34 knots during the Naval Review at Spithead. The shaft horse-power was about 2,100 and the revolutions 2,000.

Ten years later came the " Mauretania " and " Lusitania " with four propellers and a shaft horse-power of 70,000; while in H.M.S. " Hood " (1919) there are four separate sets of turbines, each driving a propeller through gearing, with a total horse-power of 144,000. At that date the total shaft horse-power of geared Parsons turbines was 17,000,000.

The following Table gives particulars of two of the latest land turbines constructed by the Company:—

Year.	Destination.	Full Load.	R.P.M.	Full Load.[1]	Steam Pressure, Gauge.	Steam Temperature.	Exhaust Vacuum.	Remarks.
		Kilowatts.		Lbs. per Kilowatt-Hour.	Lbs. per Sq. In.	F°.	Inches.	
1921	Treforest	15,000	1,500	9·10	325	725	29·10	Single cylinder.
1923	Chicago	50,000	1,800 HP. I.P. 720 L.P.	7·40	550	750	29·25	Three Cylinders (Steam reheated to 700° F. after high-pressure exhaust).

[1] Guaranteed figures.

It is common knowledge, or if not, a glance at any recent article on the subject will confirm it, that practically each step of the progress shown by these facts is based on abstract investigations; the properties of steam as it expands through a nozzle, the efficiency of various forms of nozzle, the effect of superheating, the properties of the material of the blades, questions of balancing, the form and construction of the gearing, and many other details have formed the subject of experiments, started it is true in many cases with a view to improving the performance of the turbine, but based in almost all on some fundamental law or principle of abstract science.

As in Watt's steam engine, so in the turbine, we see the work of a man " who is a born mechanic as well as a seeker after physical knowledge "; the combination of the two is required for any real advance in engineering science.

And now to conclude. There is much more that might be said, many more instances in which the discoveries of abstract science have been the foundation of the triumphs of the engineer, but I have trespassed sufficiently on your time. I have given you, I trust, sufficient examples to prove the interdependence of the two. Modern civilization is based on abstract knowledge, and all experience goes to enforce the dictum of my predecessor 30 years ago, that "the days are past when an engineer can acquit himself respectably by the aid of mother-wit alone, or of those constructive instincts" which in the past led our predecessors to such brilliant results.

Mr. BASIL MOTT said it was a very great privilege and pleasure to propose a vote of thanks to Sir Richard Glazebrook for the wonderful lecture he had delivered. He felt a little like one of the small crystals to which the lecturer had referred—somewhat distorted under pressure—and he thought nobody would venture to comment on the lecture to which the members had listened without having an opportunity for very close and careful study of it. But he ventured to think that no James Forrest Lecture of greater value had ever been given, and that the members would appreciate it to an even greater degree than they did now when they had had the opportunity of reading it. All those who had given James Forrest Lectures to The Institution were men of the highest possible distinction in their profession, and in the lines of scientific work which they had pursued; and when it was remembered what it meant to busy men to prepare a lecture of such a highly scientific character, the members should feel deeply grateful for all the trouble that had been taken, and the advantages given to the profession and to the scientific world as a whole, by Sir Richard Glazebrook. He asked the members to join in expressing a very hearty vote of thanks to the lecturer for his discourse that evening.

Sir ROBERT HADFIELD, Bart., remarked that the members had expected a brilliant lecture from Sir Richard Glazebrook, who had had opportunities such as very few members had had for carrying on research work in connection with engineering. England was indebted to him for the great work he had carried out at the National Physical Laboratory, and it was a matter of regret that his term of office had not been extended. Sir Richard Glazebrook was also greatly interested in aeronautics, and so long as there was such a man as Sir Richard in charge of that department of scientific investigation, Sir Robert Hadfield felt that this country would not be left behind by France or any other country. He was sure that in seconding the vote of thanks he was voicing the opinion of all the members when he said that Sir Richard's lecture was of the highest value. He had been delighted to hear Sir Richard refer to the important investigation of iron alloys about to be carried out under the auspices of the Institution of Mechanical Engineers. He himself had given great attention to the subject, and he was sure there was a great deal yet to be learned in regard to combinations of iron. He hoped that the research work of the Institution of Mechanical Engineers, in consultation with other Societies, would prove of very great value, and certainly the members might rely upon good work being done, as one of the chief advisers on the Committee was a gentleman for whose special knowledge in that line of work he had the highest respect, namely, Dr. Rosenhain.

The motion was carried by acclamation.

Sir RICHARD GLAZEBROOK thanked the members sincerely for the very kind way in which they had received the vote of thanks. It had been really a great pleasure and interest to him to prepare the lecture, and he had valued highly the opportunity of delivering it. It had been his high privilege recently to be made an Honorary Member of The Institution; and if in any way he had been able to show his appreciation of that honour by laying before The Institution some of the important results that had followed from the application of abstract science to engineering problems, he had only discharged a debt which had at the same time been a great pleasure.

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LONDON:
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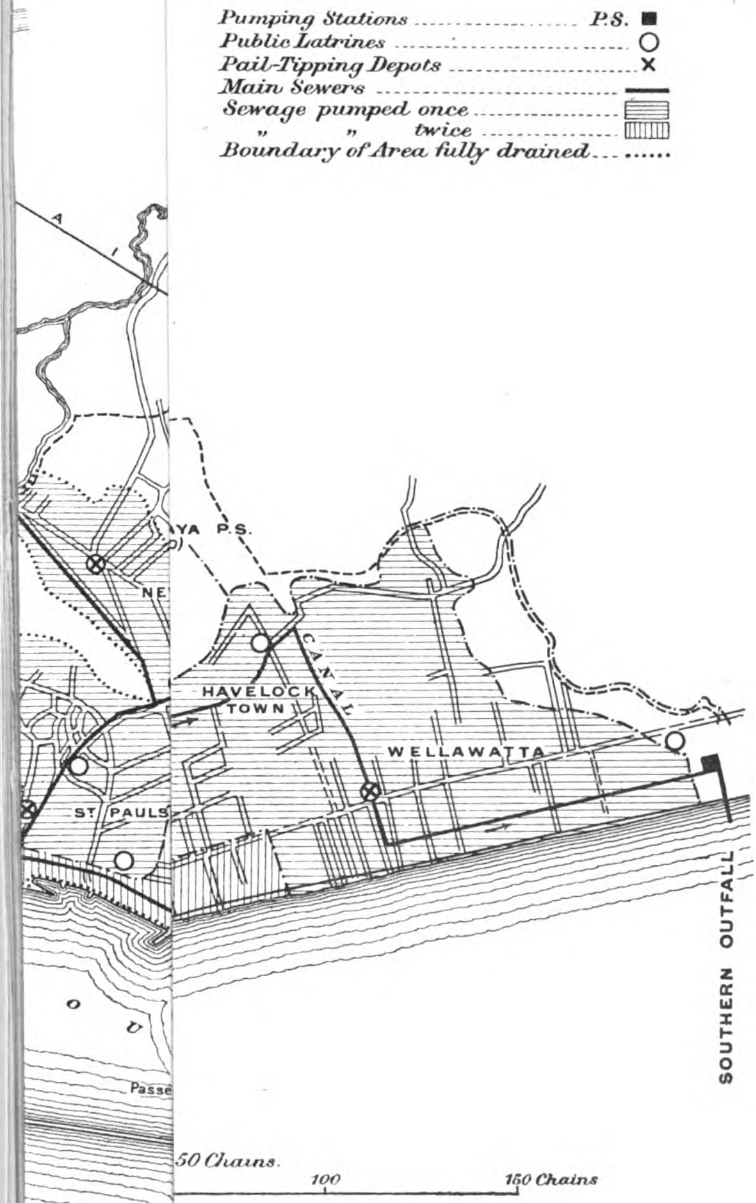
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Pail-Tipping Depots .... ×
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Sewage pumped once .... ▤
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